Design of Deep Excavations

$2 Billion Hudson Yards, New York, NY

Circular wet soil mix shaft, Florida

Cofferdams for New Tapan Zee Bridge, NY

DEEP EXCAVATION
RELIABLE GEOEXPERTISE

Feb/24/2015

Dimitrios Konstantakos, P.E.

Soil at-rest

I was sitting for a few thousand years at-rest and after a long time an excavation contractor started disturbing me. I was stressed beyond my strength, boiled up, and finally blew my excess pressure on his face. I though he knew I was more than just my SPT.

Your insitu soil

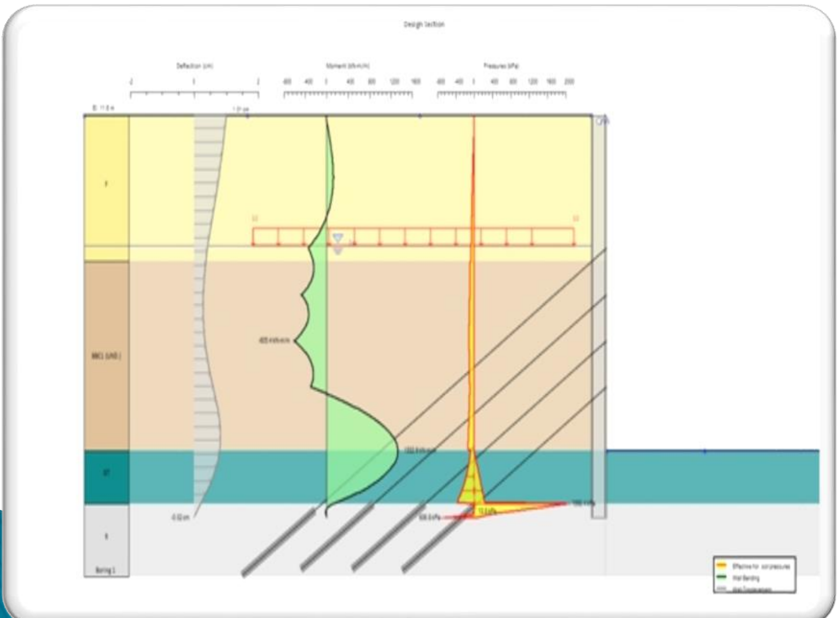
Webinar topics

- Philosophy of deep excavation design
- Identification of issues
- Understanding soil response
- Geotechnical investigations
- Wall systems
- Support systems
- Analysis methods
- Design codes
- Design examples
- Case histories

1.1 Definition – deep excavation

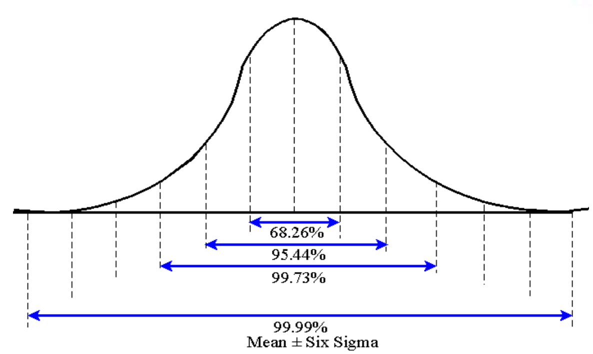
An excavation, typically deeper than 10ft (3.5m) that requires structural support.

Webinar examines vertical cut excavations that require structural support.

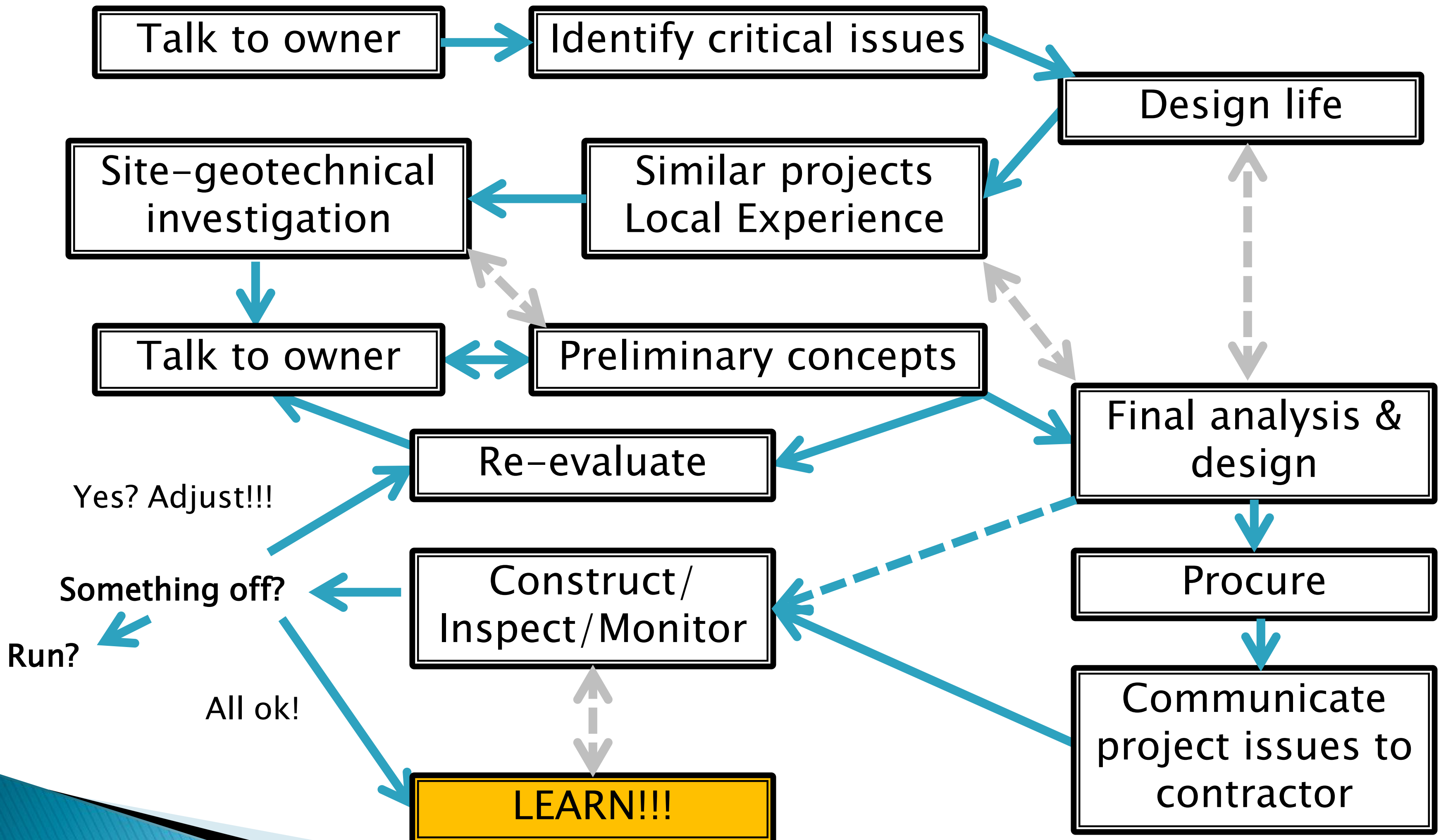


1.2 General

- A deep excavation system has to retain earth, water, and neighboring structures
- Unknown factors and risks
- Protect adjacent properties
- Design issues
- Code issues
- Economy
- Constructability

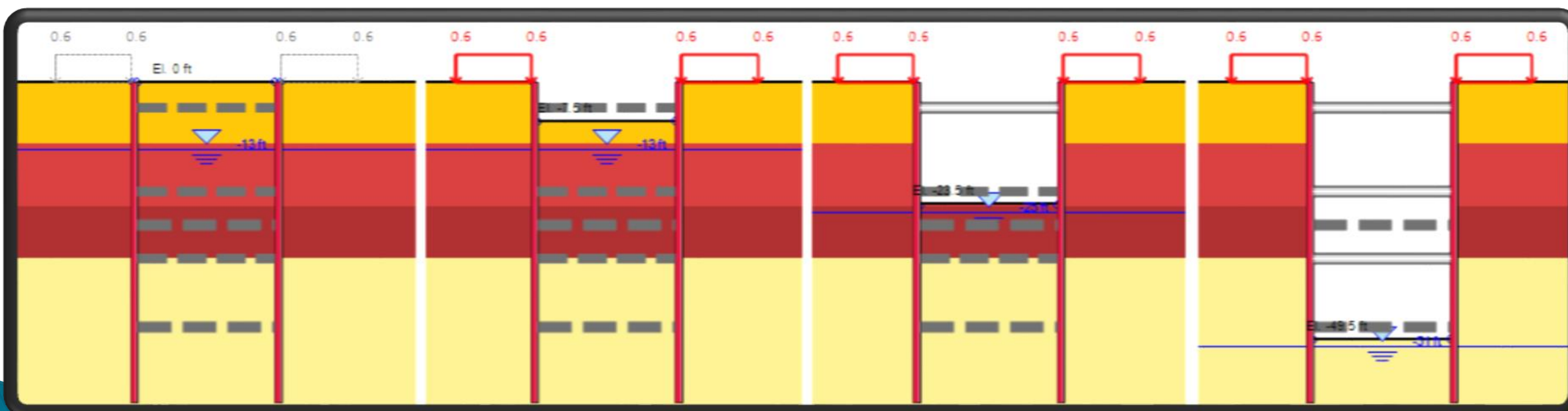


1.3 Philosophical approach



1.4 Staged construction

- Deep excavations always require staged construction.
- Even wall construction can affect performance.
- Start from at-rest conditions (or before)



2. Issues

- Soil/rock properties
- Adjacent structure condition and loads
- Design water levels
- Select appropriate earth retention system
- Examine possible failure modes
- Analysis methods
- Design/building code compliance
- Minimize deformations (wall, surface, etc)

3. Understanding soil

As engineers ask questions about your soils:

- What
- Where
- When
- How

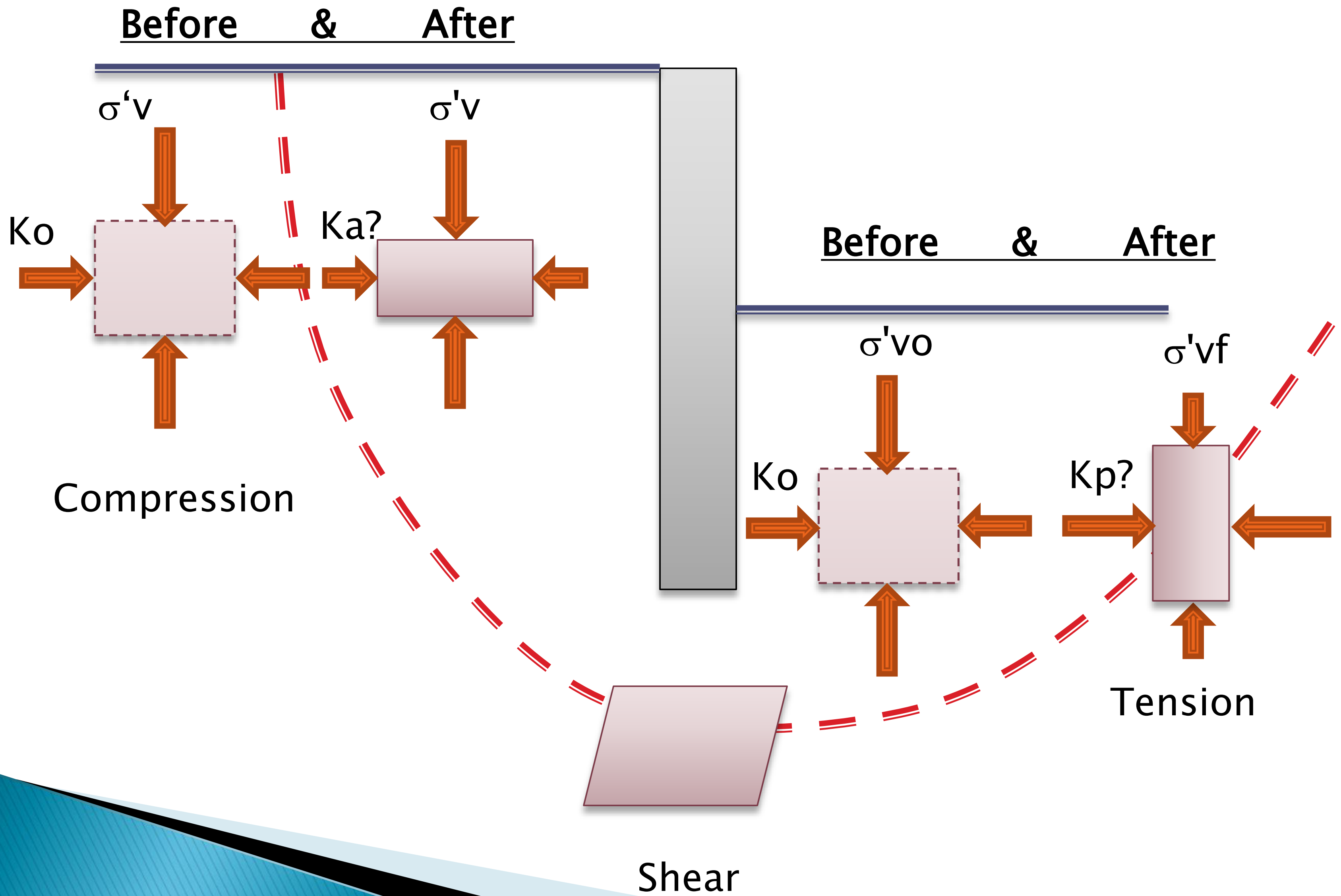
Gravel

Sand

Clayey-Silt

Clay

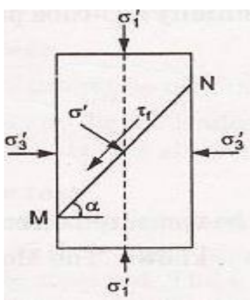
3.1 Idealized soil in excavations

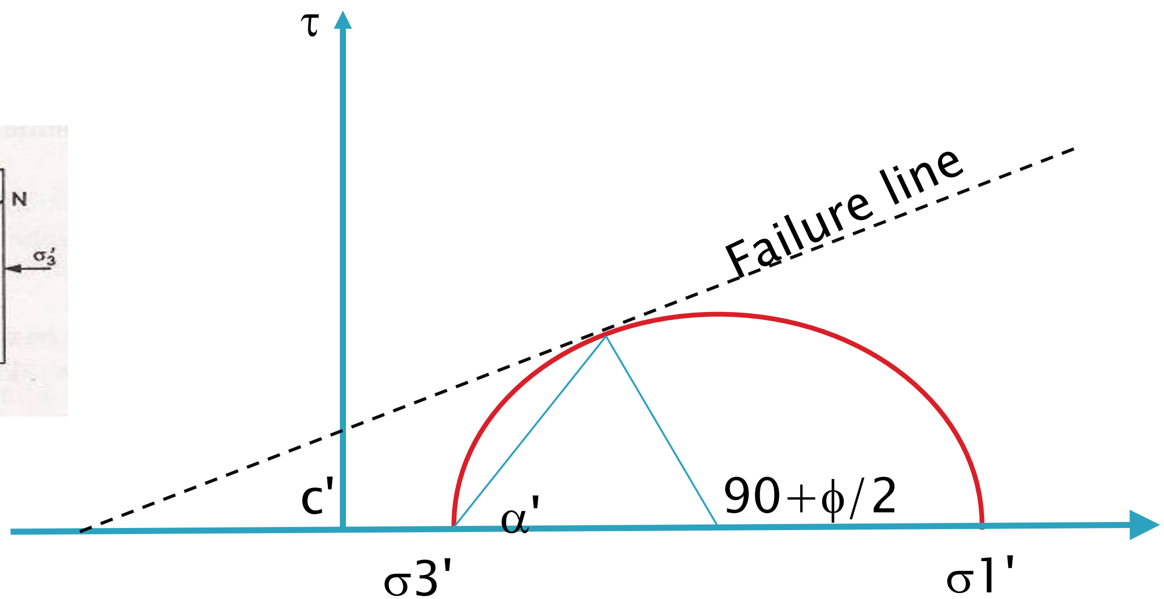


3.2 Soil shear strength

Every soil takes it's path, or stress path that is.

- Stress path
- Tension strengths <compression.
- Different response between sands and clays



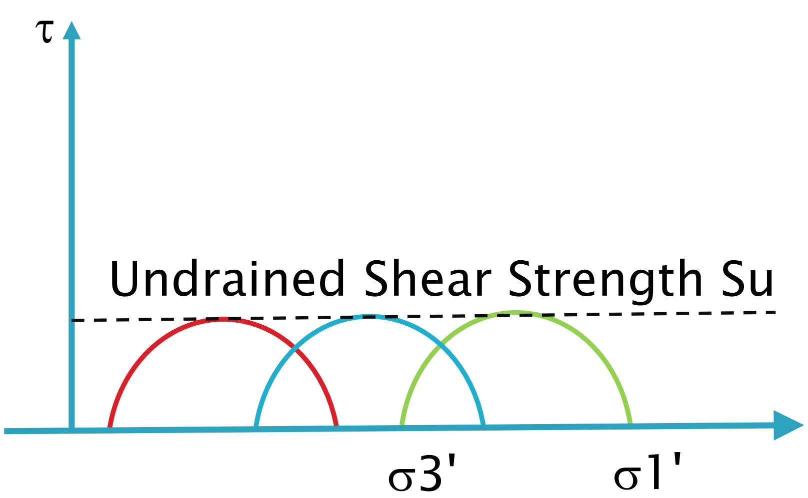


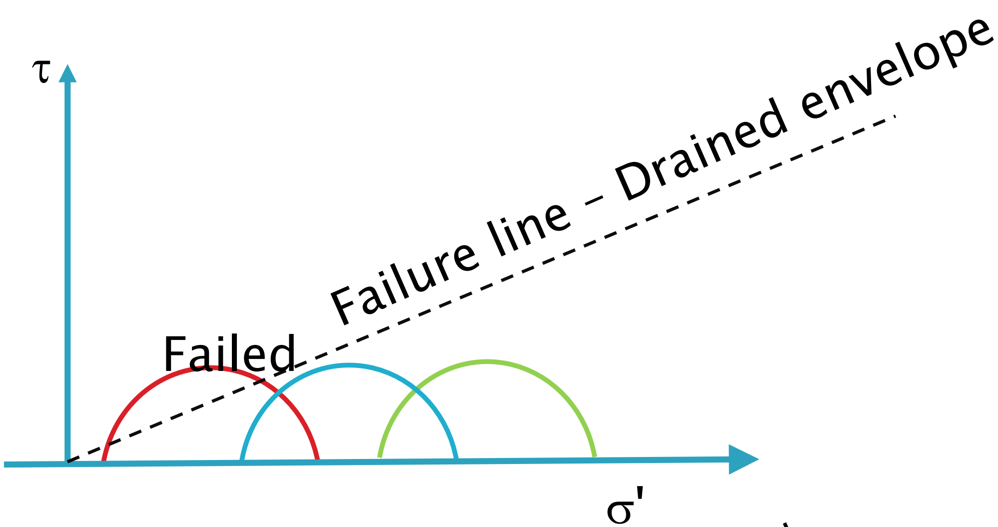
$$\tau = c' + \sigma' \tan(\phi)$$

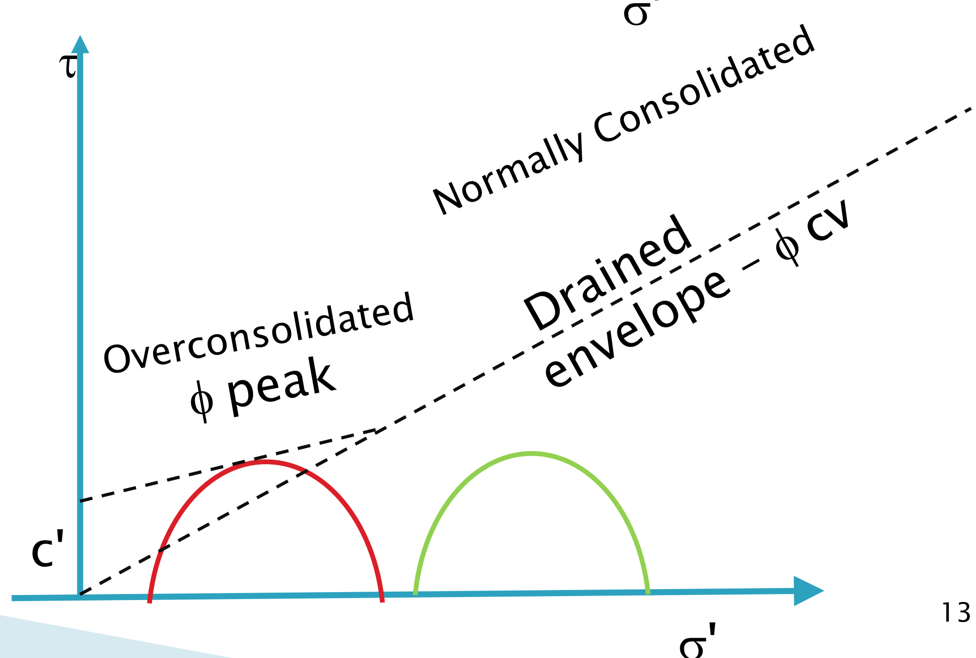
3.3 Don't clay me

- Clays are like sponges, they have absorbed so much water and they do not want to let it go.
- Clays are waterphiles, low permeability
- They resist changes in their state of stress, just like your spouse.
- So, when you are trying to excavate they are building up negative water pressures (think of it as negative emotions).
- Over time (long time) these negative pressures go away.

3.4 Clay response

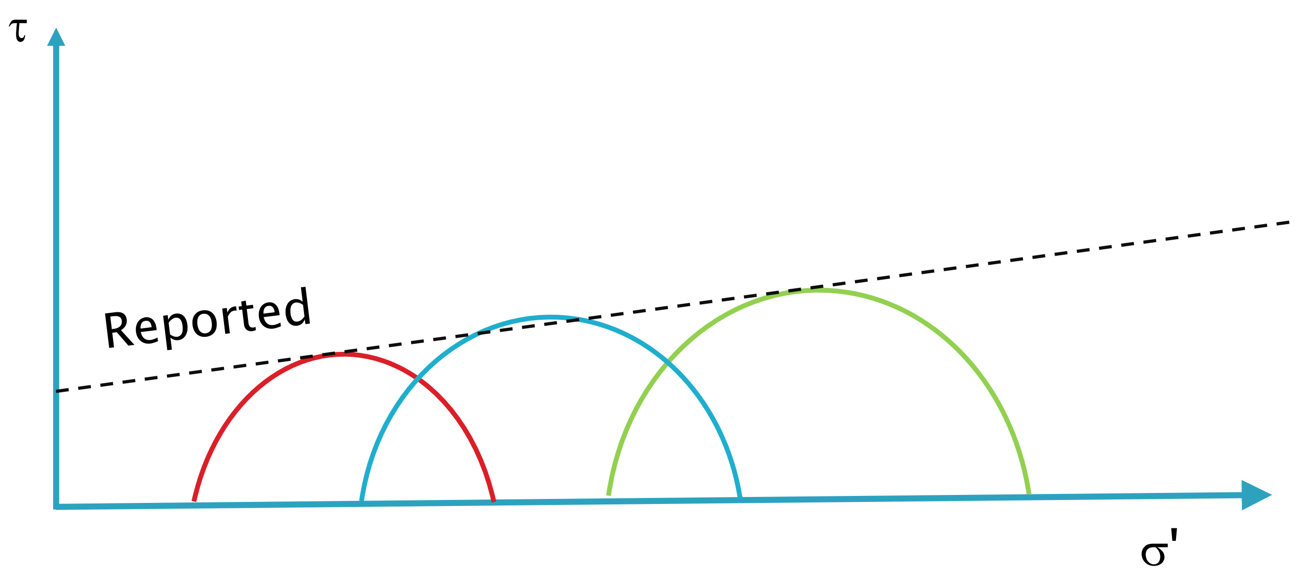






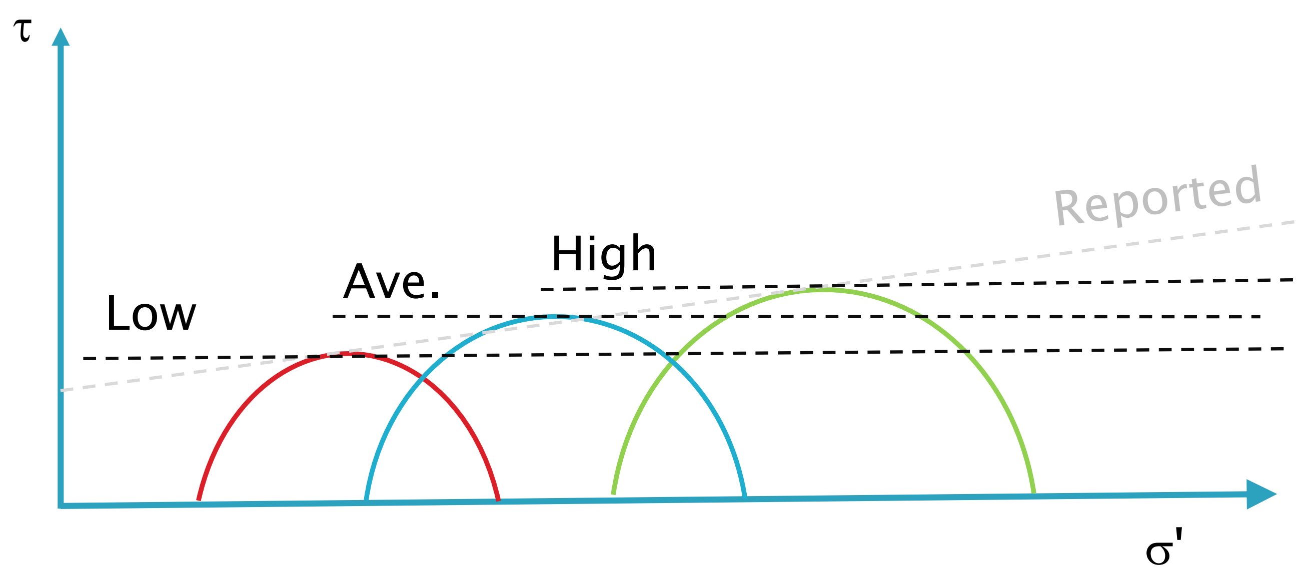
3.5.1 Quiz

- Three clay samples are taken from the same depth. They were tested in the lab and the following strengths were reported:
  - $c' = 800$ psf, $\phi = 10$ degrees



3.5.2 Quiz

- Three clay samples are taken from the same depth. They were tested in the lab and the following strengths were reported:
  - c' = 800 psf, ϕ = 10 degrees



3.6 Useful correlations

Schmertmann (1975)

$$\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_o}{p_a}\right)}\right]^{0.34}$$

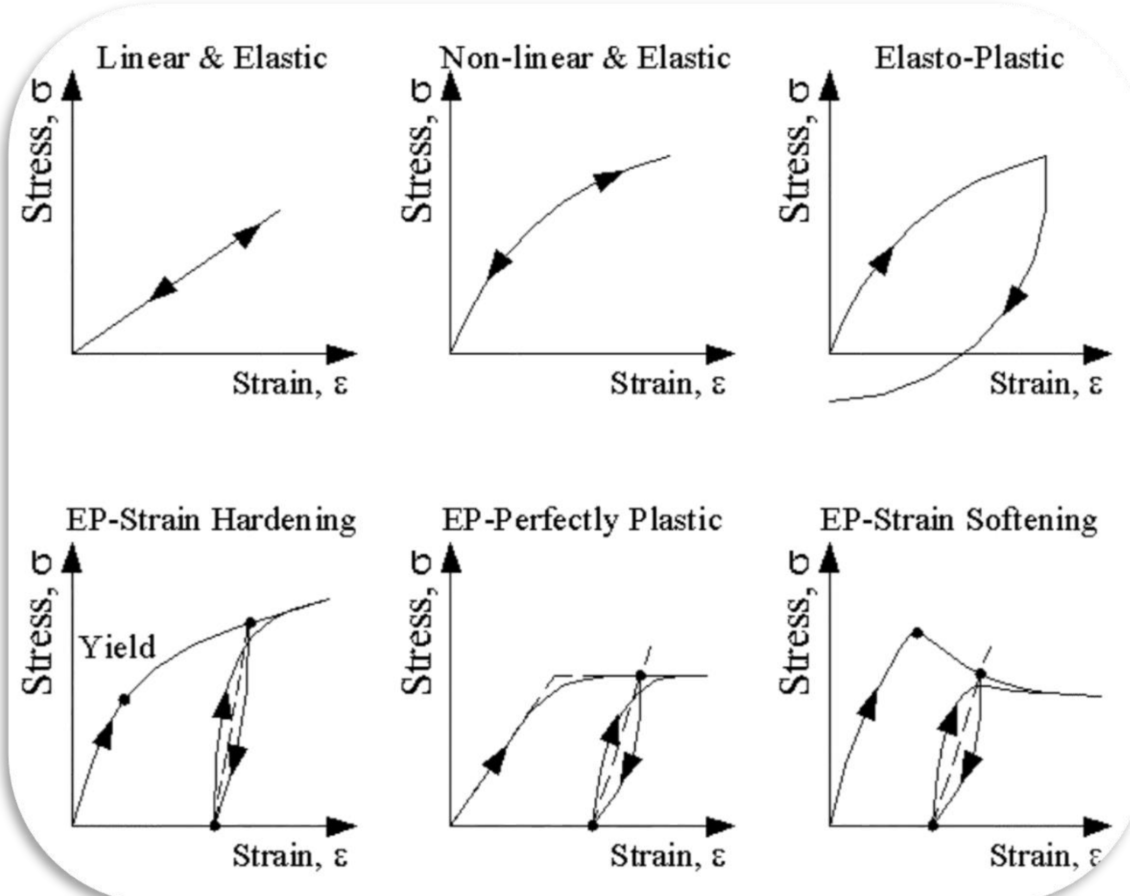
Hara et. al. (1971)

$$c_u(kPa) = 29{N_{60}}^{0.72} \qquad c_u(ksf) = 0.6{N_{60}}^{0.72}$$

Standard penetration number, N_{60}	Consistency	Unconfined compression strength, q_u (kN/m^2)
0–2	Very soft	0–25
2–5	Soft	25–50
5–10	Medium stiff	50–100
10–20	Stiff	100–200
20–30	Very stiff	200–400
>30	Hard	>400

Su = SPT/8 in ksf

3.7 Elastoplastic response



Soil inside excavation is in load-reload response
Response idealized as linear for practical purposes

E_{reload} = 3 to 5 $E_{loading}$

3.8. Rough recommendations

Recommendations by Perko

		SPT Blow Count N_{55}	Unit Weight γ (pcf)	[g/cm³]	Angle of Friction Φ (deg)	Undrained Strength s_u (psf)	[kPa]	p-y Modulus K (pci)	[kg/cm³]	Stress-Strain Modulus[1] E_s (ksi)	[MPa]	Poisson Ratio μ	Strain at 50% Peak Strength ϵ_{50}
Coarse-grain	Very loose	1-4	70	[1.12]	25	–	–	5	[0.1]	1	[9]	0.35	–
	Loose	4-10	90	[1.44]	29	–	–	25	[0.6]	2	[16]	0.35	–
	Medium	10-30	110	[1.76]	33	–	–	90	[2.4]	3	[23]	0.35	–
	Dense	30-50	120	[1.92]	39	–	–	225	[6.2]	7	[47]	0.35	–
	Very dense	>50	130	[2.08]	45	–	–	500	[13.8]	20	[137]	0.35	–
Fine-grain	Very soft	1-2	80	[1.28]	–	200	[9]	30	[0.8]	1	[7]	0.5	0.06
	Soft	2-4	85	[1.36]	–	400	[19]	100	[2.7]	2	[14]	0.4	0.02
	Medium	4-8	90	[1.44]	–	800	[38]	500	[13.8]	5	[31]	0.3	0.01
	Stiff	8-15	100	[1.6]	–	1,500	[71]	1,000	[27.6]	7	[47]	0.2	0.005
	Very stiff	>15	120	[1.92]	–	3,000	[143]	2,000	[55.3]	10	[71]	0.1	0.003
Weathered bedrock	Soft	<30	120	[1.92]	–	4,000	[191]	2,000	[55.3]	70	[482]	0.25	0.003
	Medium	30-50	130	[2.08]	–	10,000	[478]	3,000	[83]	280	[1931]	0.25	0.002
	Hard	50-100	135	[2.16]	–	20,000	[957]	4,000	[110.7]	520	[3586]	0.25	0.001
	Very hard	>100	140	[2.24]	–	50,000	[2394]	5,000	[138.4]	700	[4828]	0.25	0.001

Always take tables with a grain of salt,
In this table unit weights are conservative for piles
but not for excavations

3.9 Geotechnical/site investigation

- Importance of site visit
- Relevant information (historic, geologic, etc)
- Identify code requirements
- Identify required tests (insitu/lab)
- Go beyond SPT's
- Determine/monitor groundwater levels
- Identify depth of investigations (consider increased excavation requests).
- Realistic conservative estimates.

3.9.1 Borehole depths/locations

- Critical locations
- Next to buildings/structures
- Extend beyond excavation (1.5 x Hexc)
- 3m in rock
- Minimum code requirements (NYC incoming revisions one borehole/50ft)

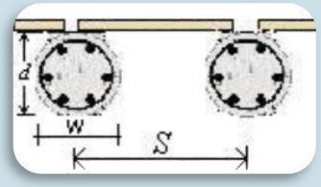
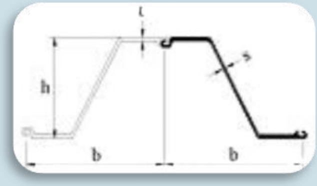
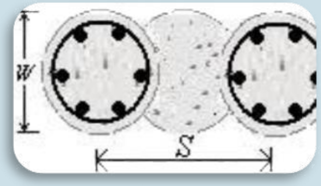
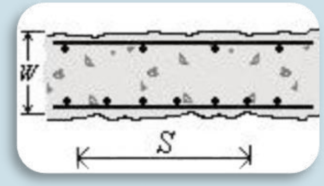
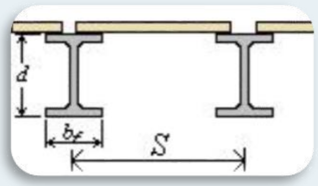
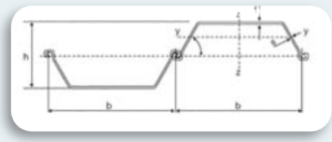
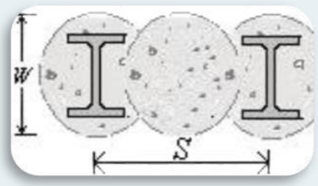
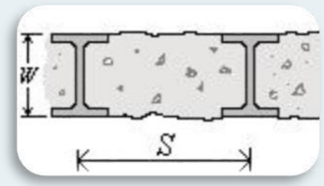
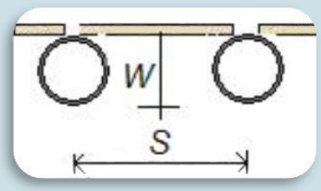
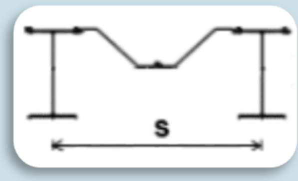
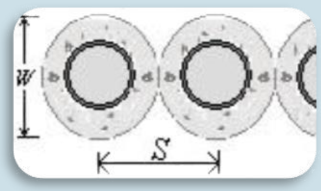
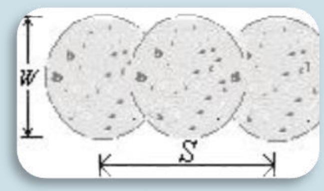
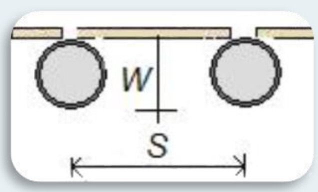
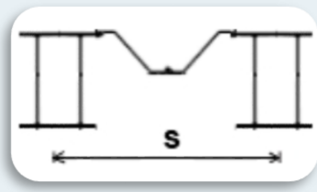
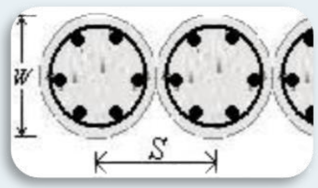
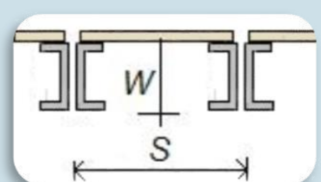
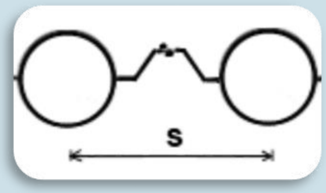
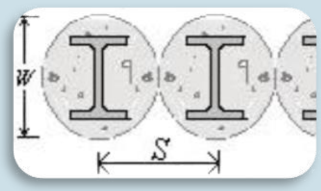
3.10 Useful tips

- A little cohesion goes a long way
- Be considerate of soil variability
- Look out for spacial variability
- Look out for problematic soils (running silts, soft organics, normally consolidated soft clays, fissured clays).
- What is this clay doing on this mountain (hill)?
- Draw your soil profile sections along the excavation.

4. Wall Systems

A wall is the main structural system that provides earth retaining support. With the exception of cantilever walls and some circular shafts most walls require bracing.

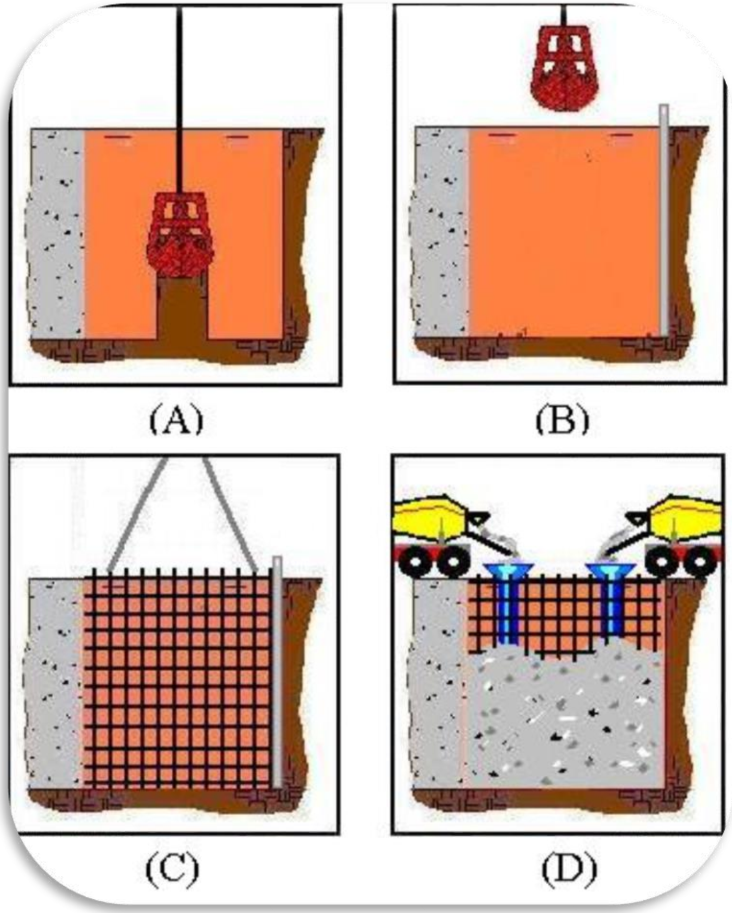
- Temporary/Permanent
- Drilled/Cast-in place/Driven/Soil mix
- Flexible/rigid
- Watertight/permeable

Soldier pile walls	Sheet piles / Combined walls	Secant/Tangent piles	Slurry walls SPTC, Soil Mix, etc
			
			
			
			Jet grout
			

Soldier pile walls

Sheet pile walls

Slurry walls / Soil mix



5. Support systems

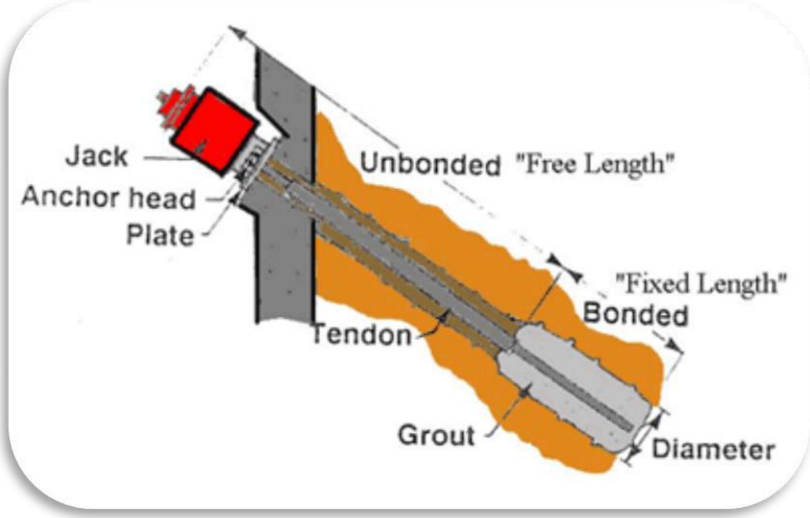
Supports provide lateral bracing for walls.

- Temporary/permanent
- Active or passive
- Internal or external

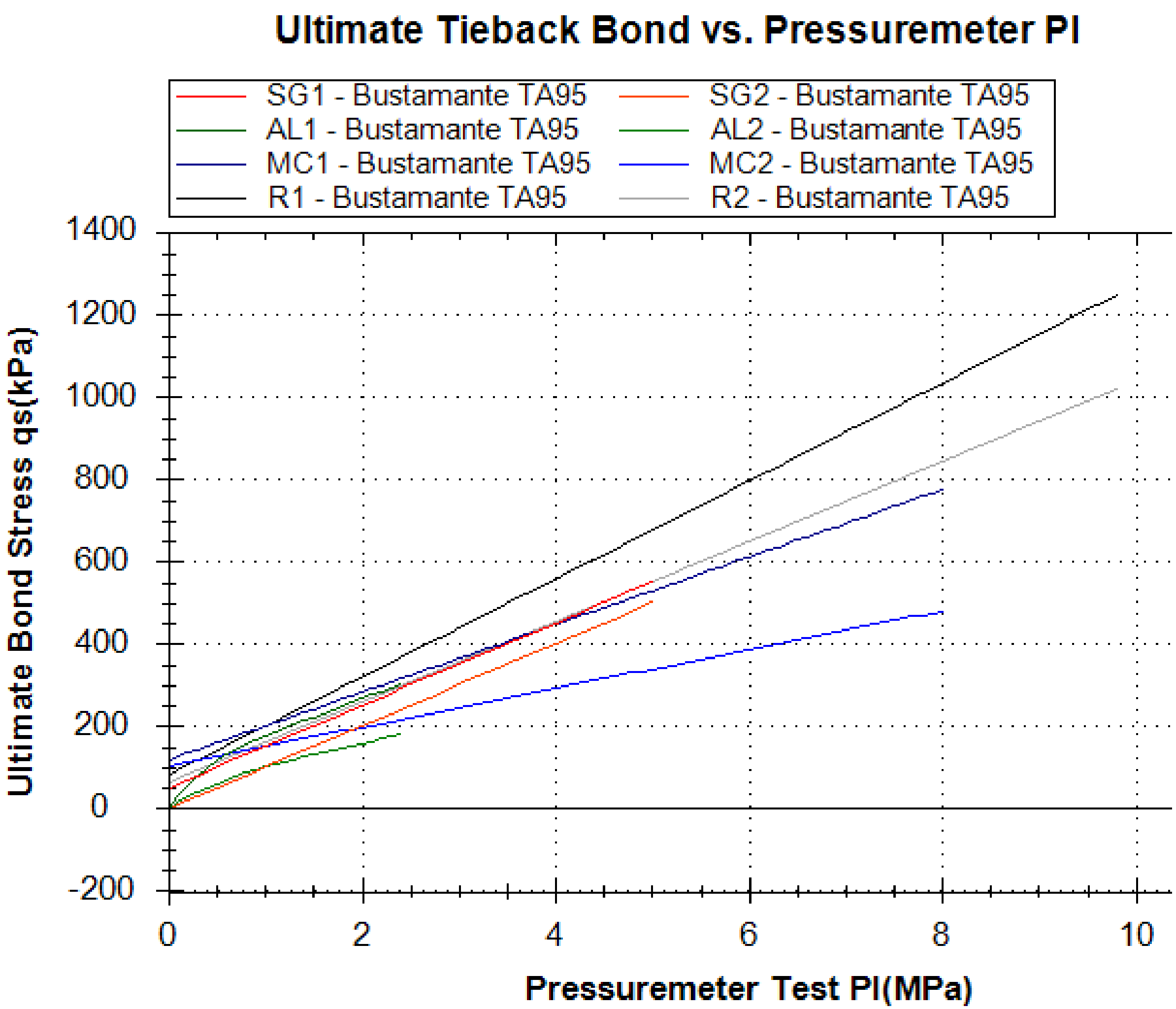
Type	Prestressed	Internal /External	Temporary/ Permanent
Tiebacks	Yes	External	Both
Steel struts	Some times	Internal	Temporary
Deadman	No	External	Both
Rakers/Heelblocks	No	Internal	Temporary
Top/Down	No	Internal	Permanent

5.1 Tiebacks (ground anchors)

- Angle inclination
- Locate beyond active wedge (below excavation, +0.1 to 0.2 Hexc)
- Design life/corrosion
- Stress relaxation with time



5.1 Bond resistance & pressuremeter test



SG1, AL1:etc graphs for IRS technique (multiple injection, pressure grouted anchors with pressure >= PL, tube a manchettes technique).

SG2, AL2:etc graphs for IGU technique (single injection, gravity grouted anchors with single pressure between PL/2 and PL).

SG1, SG2= Sands and gravels.

AL1, AL2= Silts and clays.

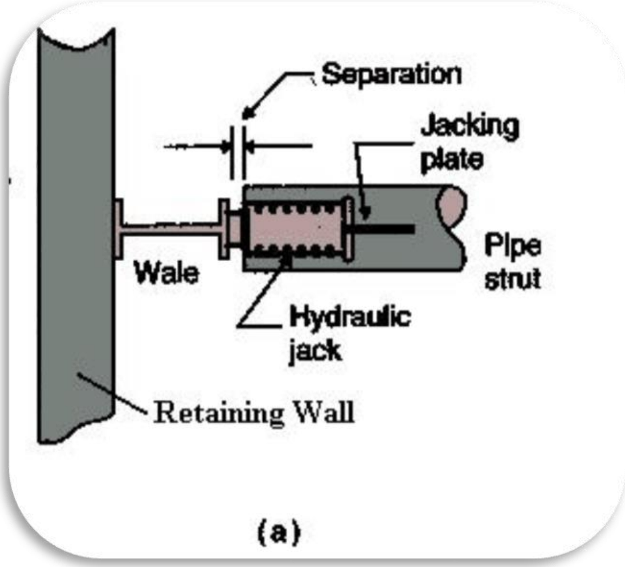
MC1, MC2= Chalk-Marl, Calcareous Marl rock altered (Craie Marne, + Marno-Calcaire)

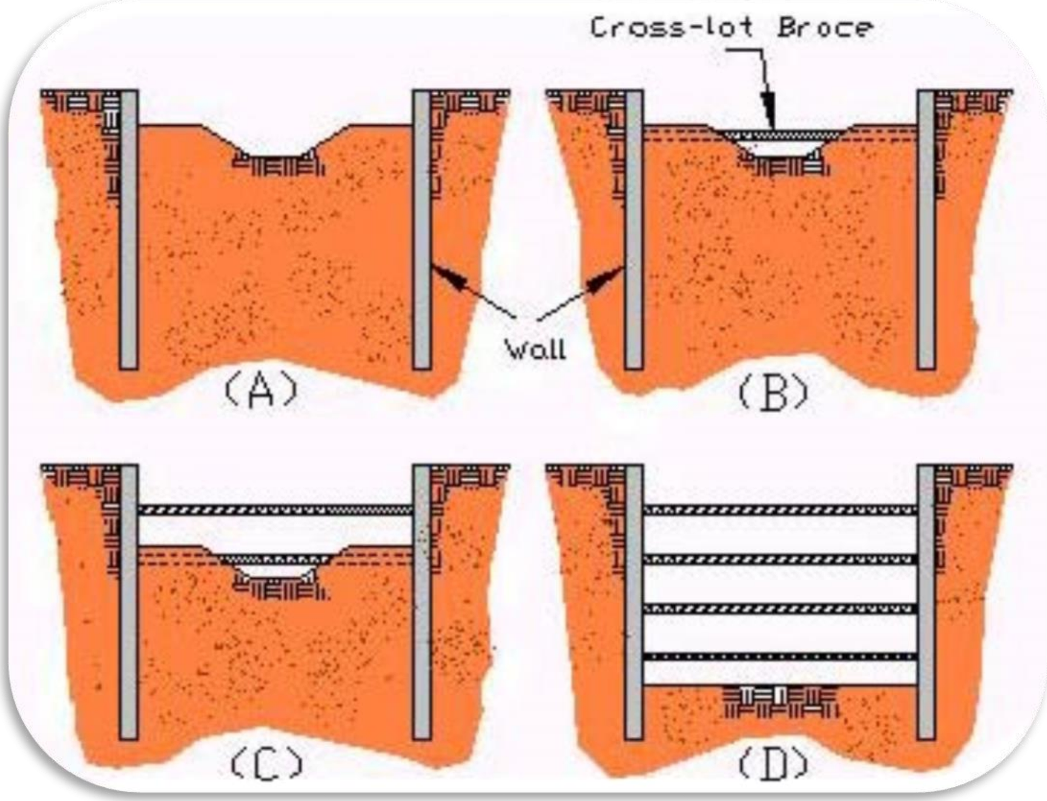
R1, R2= Altered or decomposed rock

PL = Pressuremeter limit.

IRS technique French standards allow the assumption of a greater grouted body diameter. This effect can only be accounted by increasing the Dfix diameter in each ground anchor.

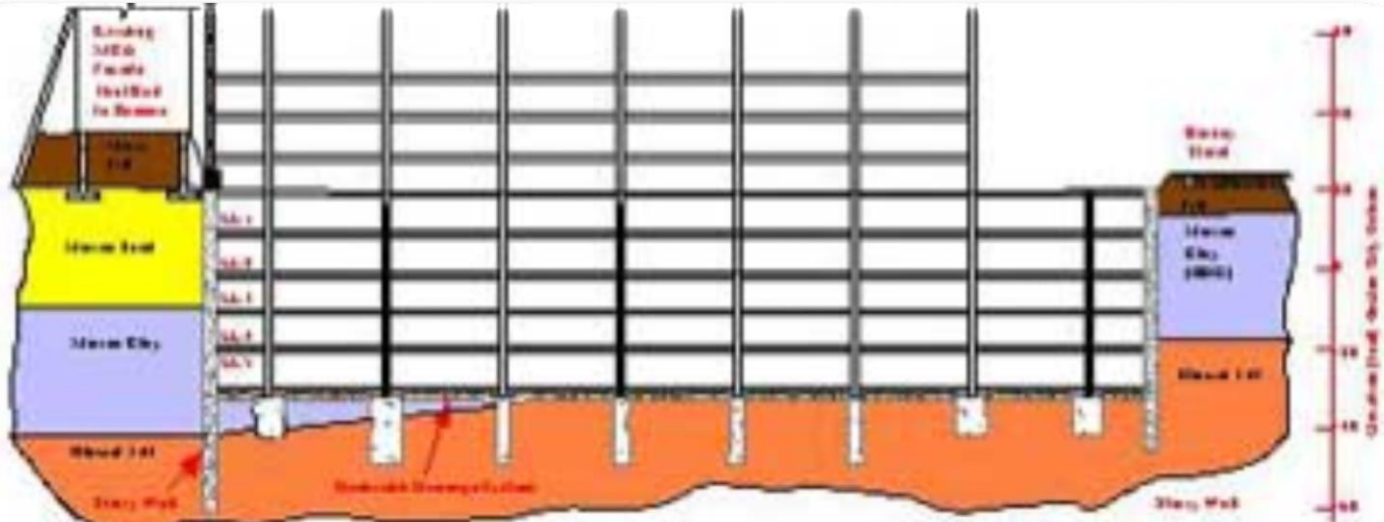
5.2 Steel struts





5.2 Steel struts/Internal Bracing

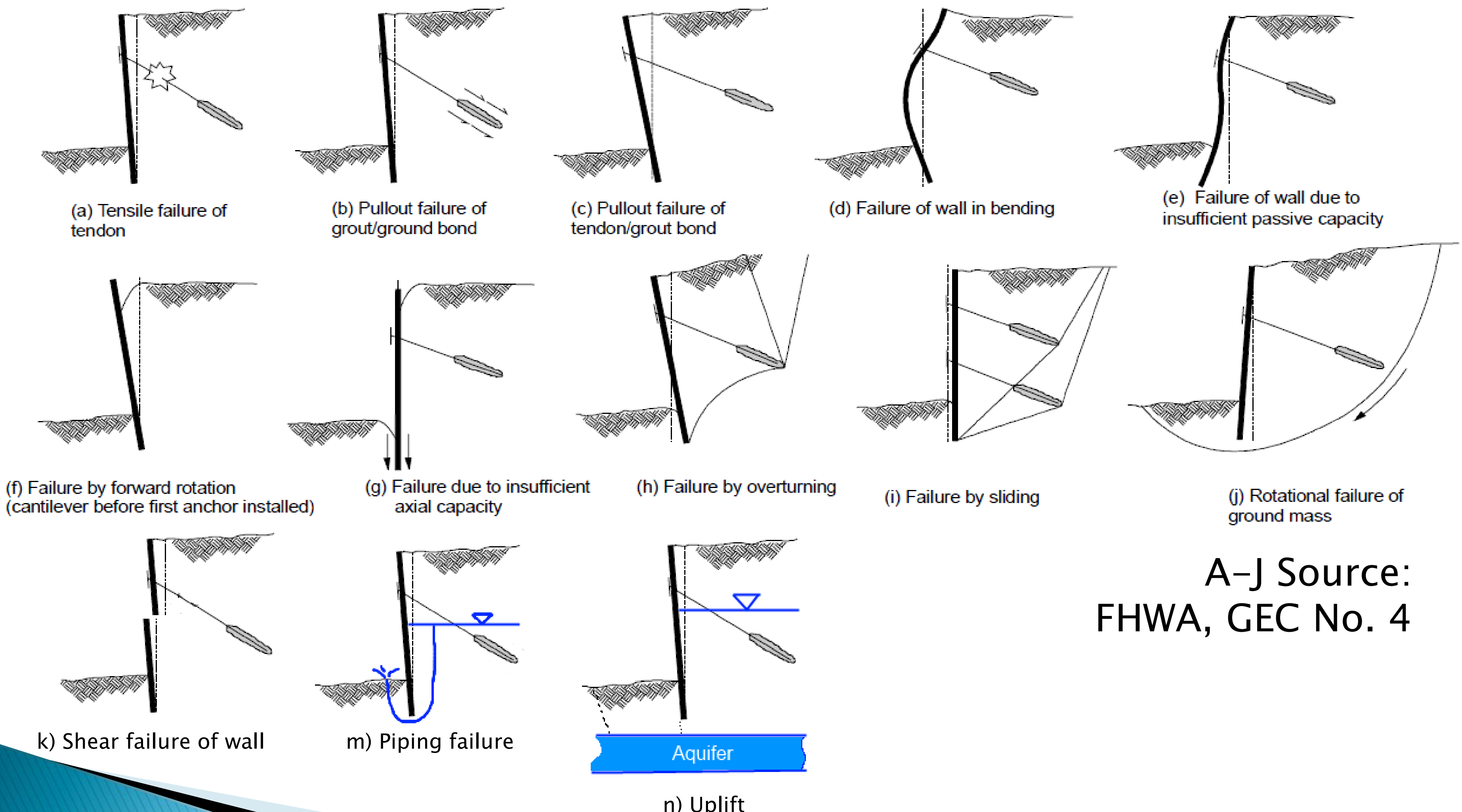
5.3 Top/down construction



d) Excavation & lowest basement floor completed

Excavating Under The Grade Level Slab

6. Failure modes (not all)



A–J Source:
FHWA, GEC No. 4

6. Failures

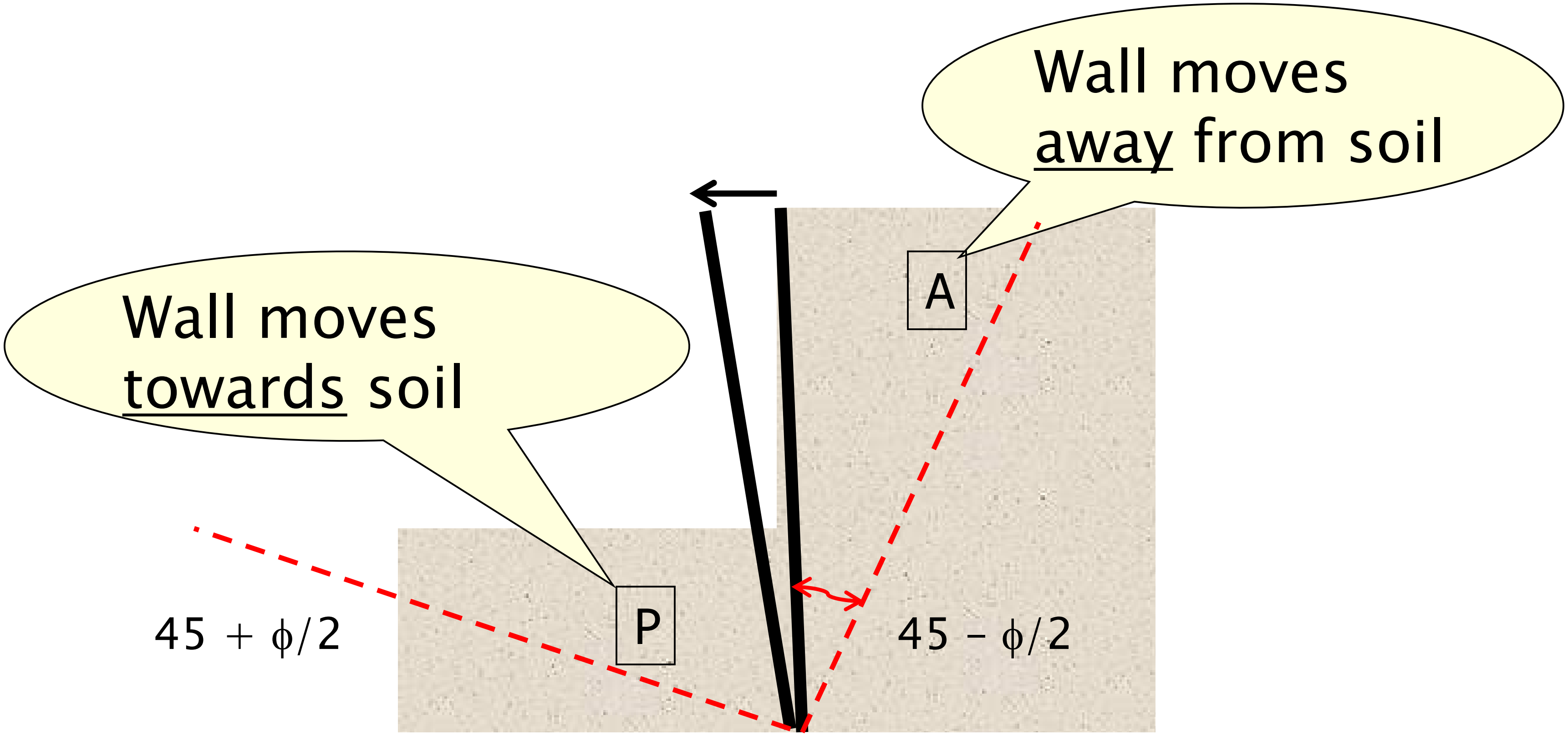
7. Analysis methods

Analysis methods used to determine support and wall forces, displacements, and other important behavior data.

All analysis methods are simplifications of very complex interaction problems.

Each analysis methods has advantages and disadvantages.

7.1 Active and passive



*Frictional soils only

7.1 Lateral earth pressure coefficients

$$[\sigma_h']_{active} = K_A\sigma_v' - 2c\sqrt{K_A}$$

* Assumes smooth wall

$$[\sigma_h']_{passive} = K_P\sigma_v' + 2c\sqrt{K_P}$$

*Only vertical walls

a) Rankine passive earth pressure coefficient: $Kp = \frac{(1+\sin(\varphi))}{(1-\sin(\varphi))}$

---

b) Coulomb passive earth pressure coefficient: $Kph = Kp \cdot cos(\delta - \theta)$

$$Kp = \frac{cos^2(\varphi + \theta - \bar{\beta})(1-ay)}{cos^2(\theta)\ cos^2(\bar{\beta}) cos(\delta - \theta + \bar{\beta}) \left[1 - \sqrt{\frac{sin(\delta+\varphi) sin(\varphi + \alpha - \bar{\beta})}{cos(\delta - \theta + \bar{\beta}) \cos(\alpha - \theta)}}\right]^2}$$

α= Slope angle (positive upwards)

$\bar{\beta}$= Seismic effects $= tan^{-1}(\frac{ax}{1-ay})$ with

ax = horizontal acceleration (relative to g)

ay= vertical acceleration, +upwards (relative to g)

θ= Wall angle from vertical (0 radians wall face is vertical)

---

c) Lancellotta: According to this method the passive lateral earth pressure coefficient is given by:

$$Kph = Kpe \cdot \bar{\gamma} \cdot cos(\alpha - \bar{\beta})$$

$$Kpe = \left[\frac{cos(\delta)(\cos(\delta) + \sqrt{sin^2\varphi - sin^2(\delta)})}{cos(\alpha - \bar{\beta}) - \sqrt{sin^2\varphi - sin^2(\alpha - \bar{\beta})}}\right] \cdot e^{2\vartheta \tan(\varphi)}$$

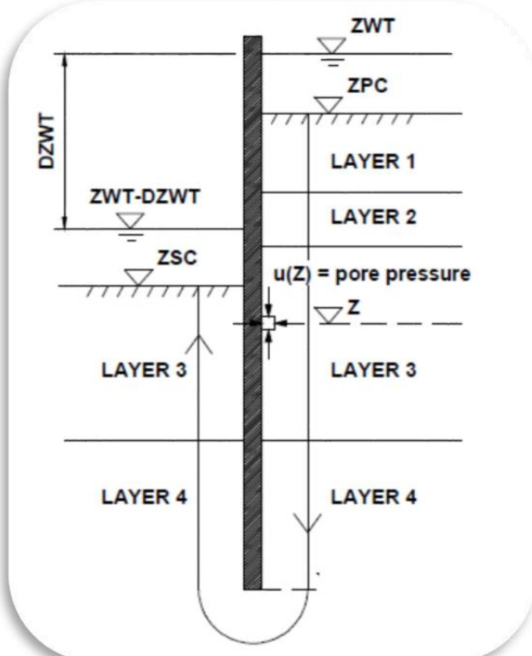
$$\bar{\gamma} = \sqrt{(1-ay)^2 + (1+ax)^2}$$

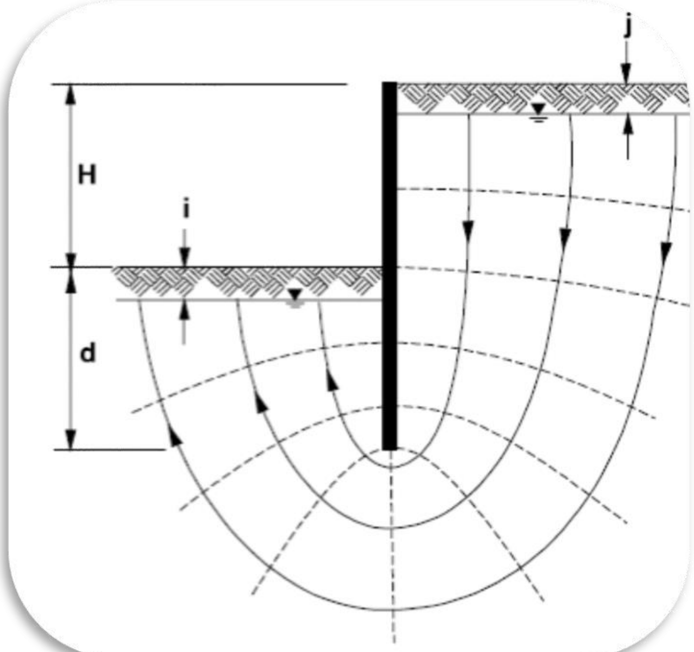
$$2\vartheta = sin^{-1}\left(\frac{\sin\delta}{\sin\varphi}\right) + sin^{-1}\left(\frac{\sin(\alpha - \bar{\beta})}{\sin\varphi}\right) + \delta + (\alpha - \bar{\beta}) + 2\bar{\beta}$$

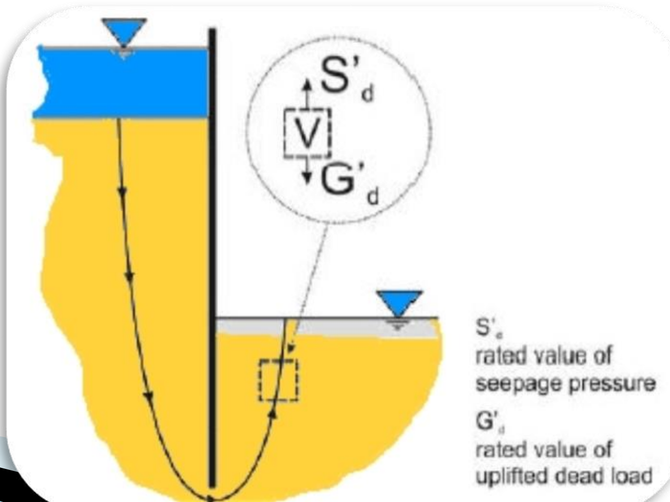
7.1 Lateral earth pressure coefficients

	Coulomb	Caquot-Kerisel	Lancellotta
Failure surface	Wedge	Log-spiral	Log-spiral
Wall friction	Yes	Yes	Yes
Correlation	Equation	Tables	Equation
Ka	Yes	Yes	No
Kp	Yes	Yes	Yes
Seismic	Yes	No	Yes

7.2 Water


ZWT
ZPC
DZWT
LAYER 1
LAYER 2
ZWT-DZWT
ZSC
u(Z) = pore pressure
Z
LAYER 3
LAYER 3
LAYER 4
LAYER 4


j
H
i
d


S'd
V
G'd
S'd
rated value of
seepage pressure
G'd
rated value of
uplifted dead load

7.3 Analysis methods

- Conventional methods
- Beam on elastoplastic foundations
- Finite elements/Finite difference
- Neural networks

	Conventional Methods	Beam on elastic foundations	Finite-elements
Easy to check	Yes	Yes/No	No
SSI	No	Yes	Yes+
Simple input	Yes	Yes/No	No
Time	Hand calculations	Faster	Fast
Realistic behavior	?	?	?

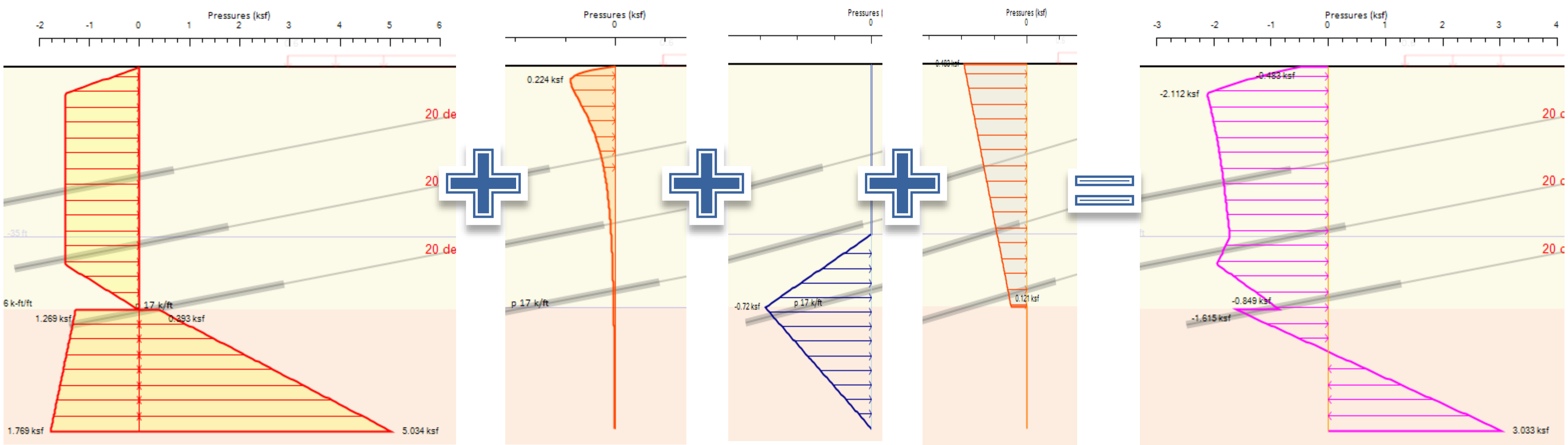
7.3 Conventional methods

General

- Assume lateral earth pressures.
- Determine fixity locations for forces at subgrade.
- Analyze wall beam with assumed loads.

- Advantages: Easy method to verify. Gives a back check for more rigorous methods.
- Disadvantages: Soil-structure interaction ignored.

7.3.1 Determine net loading diagram on wall



Soil pressures Surch. Water Seismic Net loading

Soldier pile walls (berlin type), 3D effects
Pile spacing above excavation,
Active and passive effective widths, Water width

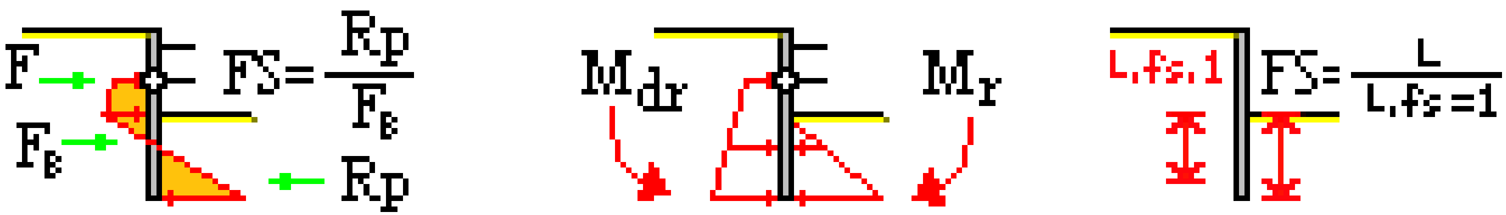
7.3.2 Wall embedment safety factors (limit-equilibrium)

- Horizontal force
- Moment
- Length

$$FSpas = \frac{Available\ Resistance\ beneath\ \mathrm{virtual}\ fixity\ point}{Hor.reaction\ at\ virtual\ point + driving\ pressures\ beneath\ virtual\ fixity\ point}$$

$$FSrotation = \frac{Resisting\ moments\ about\ a\ point}{Driving\ moments\ about\ the\ same\ point}\ (Eq.\ 9.2)$$

$$FSembed = \frac{Available\ wall\ embedment\ depth}{Max.Required\ embedment\ depth\ for\ FS = 1\ from\ Equations\ 1\&\ 2\ above}\ (Eq.\ 9.3)$$



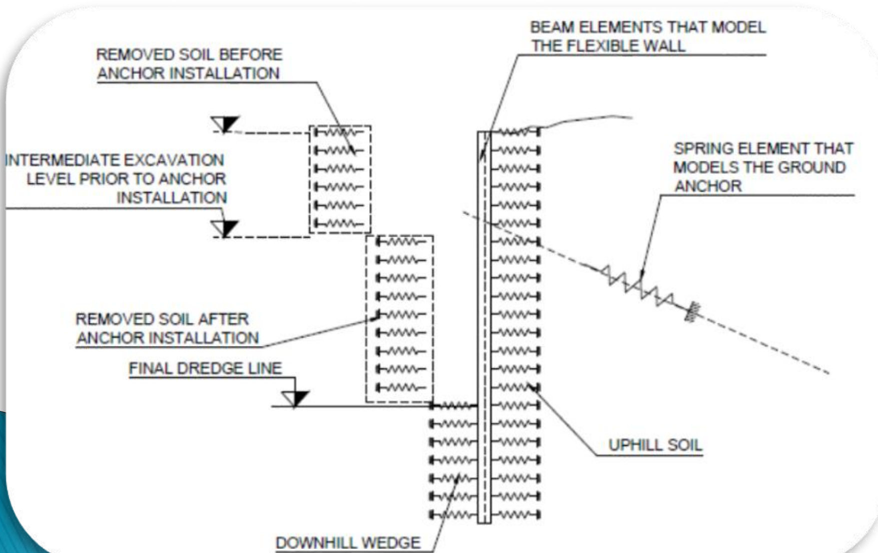
7.4.1 Beam on elastic foundations

Soil assumed as elastic (elastoplastic) springs.

Different methods available:

a) Driving pressures assumed, passive springs
b) Active and passive soil springs
c) Stage dependency?

- Subgrade reaction (depends on dimensions)
- From soil elasticity with active/passive wedges



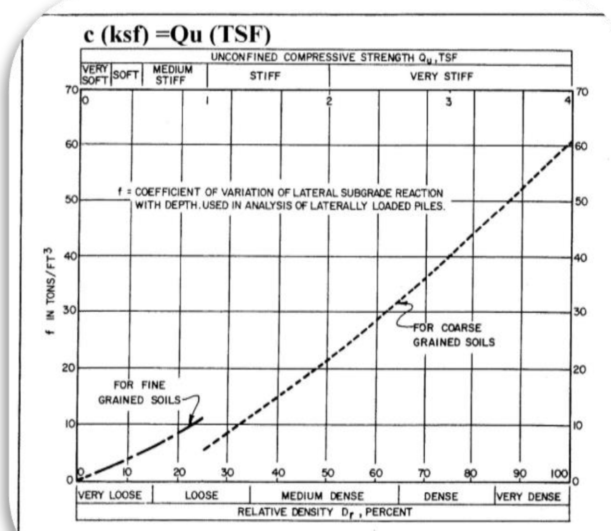
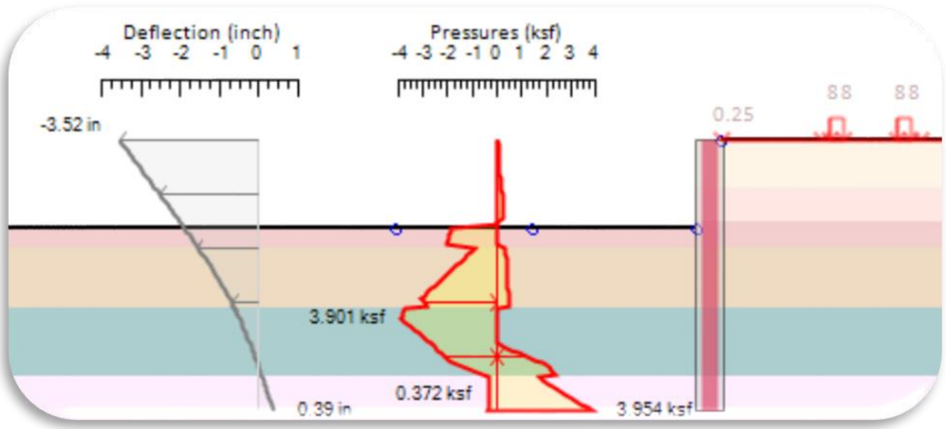


FIGURE 9
Coefficient of Variation of Subgrade Reaction

7.4.2 Wall embedment safety

$$FSpas.mob = \frac{Available\ soil\ passive\ resistance\ beneath\ subgrade}{Mobilized\ passive\ soil\ reaction\ beneath\ subgrade} \ (Eq.9.4)$$

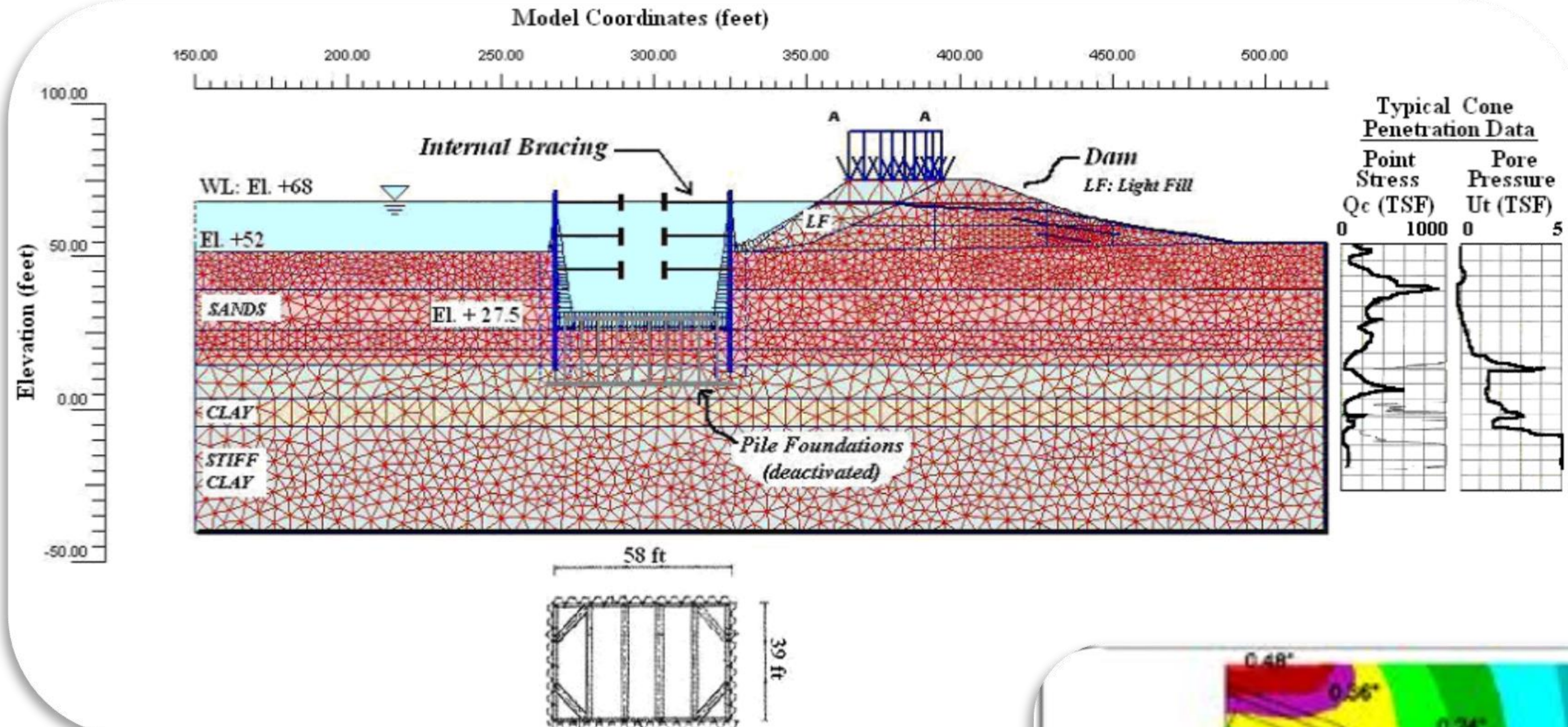


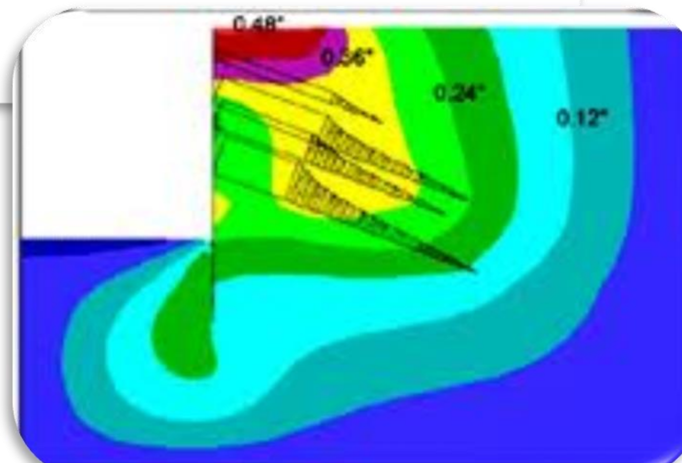
7.5.1 Finite element analysis

- Discretize soil in simple elements
- Boundary conditions
- Model soil with strength and elasticity
- Model structures
- Include construction stage history

- Advantages: Full soil structure interaction
- Disadvantages: Requires skilled designer, difficult to verify

7.5.2 Finite element analysis





$$\begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \gamma_{xx} \\ \gamma_{yy} \\ \gamma_{zz} \end{bmatrix} = \frac{1}{E} \begin{bmatrix} 1 & -v & -v & 0 & 0 & 0 \\ -v & 1 & -v & 0 & 0 & 0 \\ -v & -v & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 2(1+v) & 0 & 0 \\ 0 & 0 & 0 & 0 & 2(1+v) & 0 \\ 0 & 0 & 0 & 0 & 0 & 2(1+v) \end{bmatrix} \begin{bmatrix} \sigma'_{xx} \\ \sigma'_{yy} \\ \sigma'_{zz} \\ \sigma'_{xy} \\ \sigma'_{yz} \\ \sigma'_{zx} \end{bmatrix}$$

E= Young's modulus v= Poisson's ratio

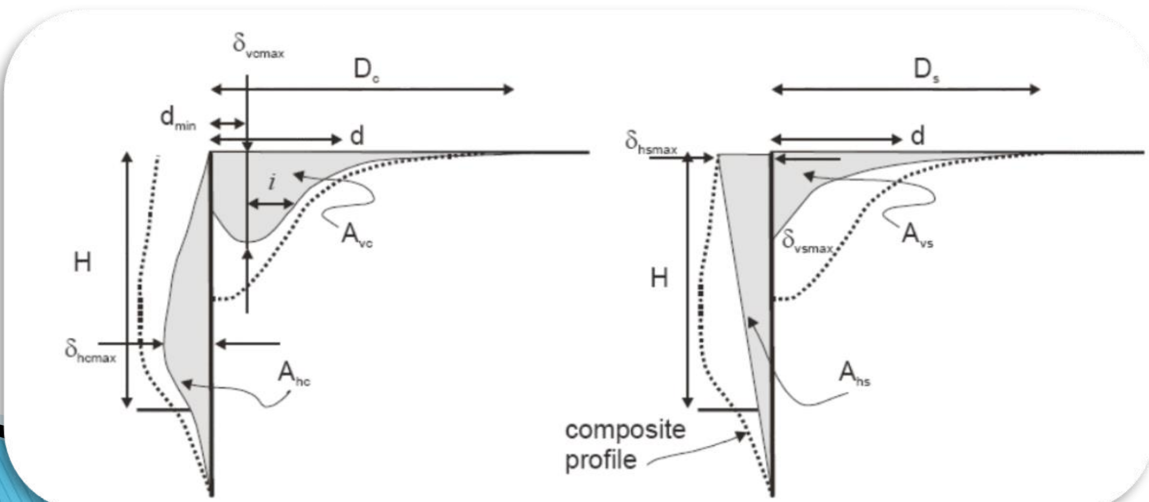
σ'_{xx} = Effective stress

7.5.3 Finite element issues

- GIGO (Garbage in – garbage out)
- It is good to know what to expect!
- Small strain stiffness vs. large strain
- Basal heave and cantilever displacements usually overestimated
- Surface settlements occasionally are out of touch (models without anisotropy)
- Nice colors can give a false sense of assurance

7.6 Vertical settlement

- Horizontal wall movement
- Wall construction
- Ground anchor construction (soil loss)
- Vibration induced
- Consolidation
- Dewatering



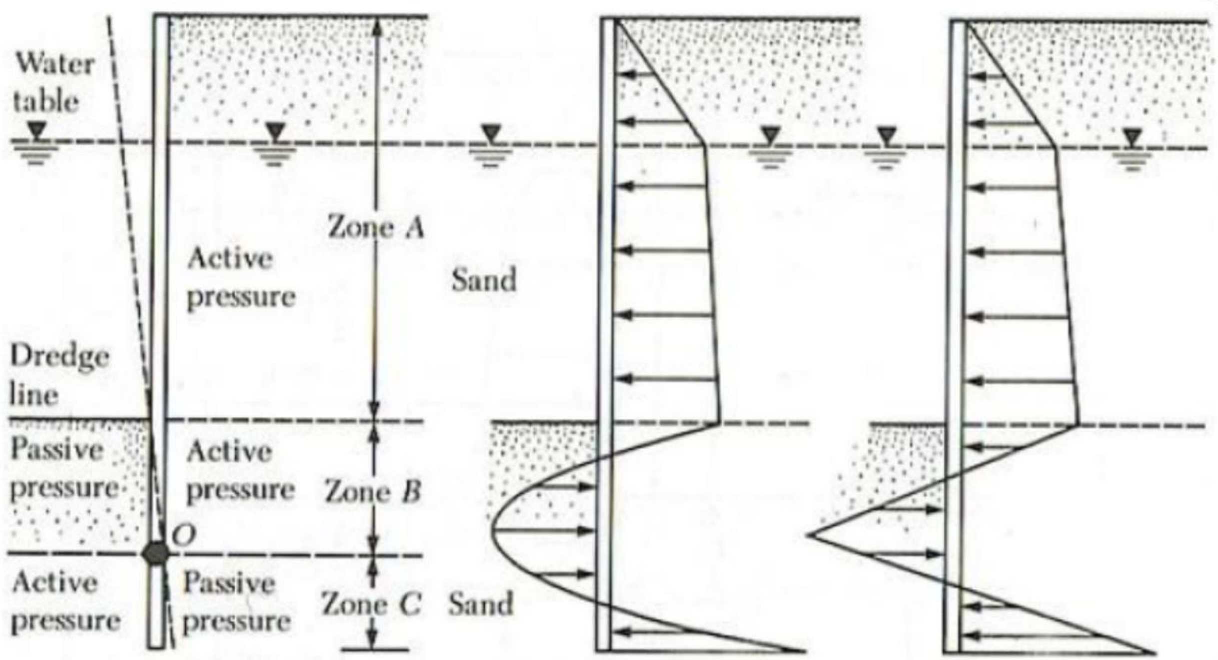
7.7 Other issues

- 3D arching effects
- Thermal loads on steel struts
- Shrinkage issues on concrete slabs
- Connection details
- Pin piles for struts
- System redundancy

8.1 Cantilever wall analysis

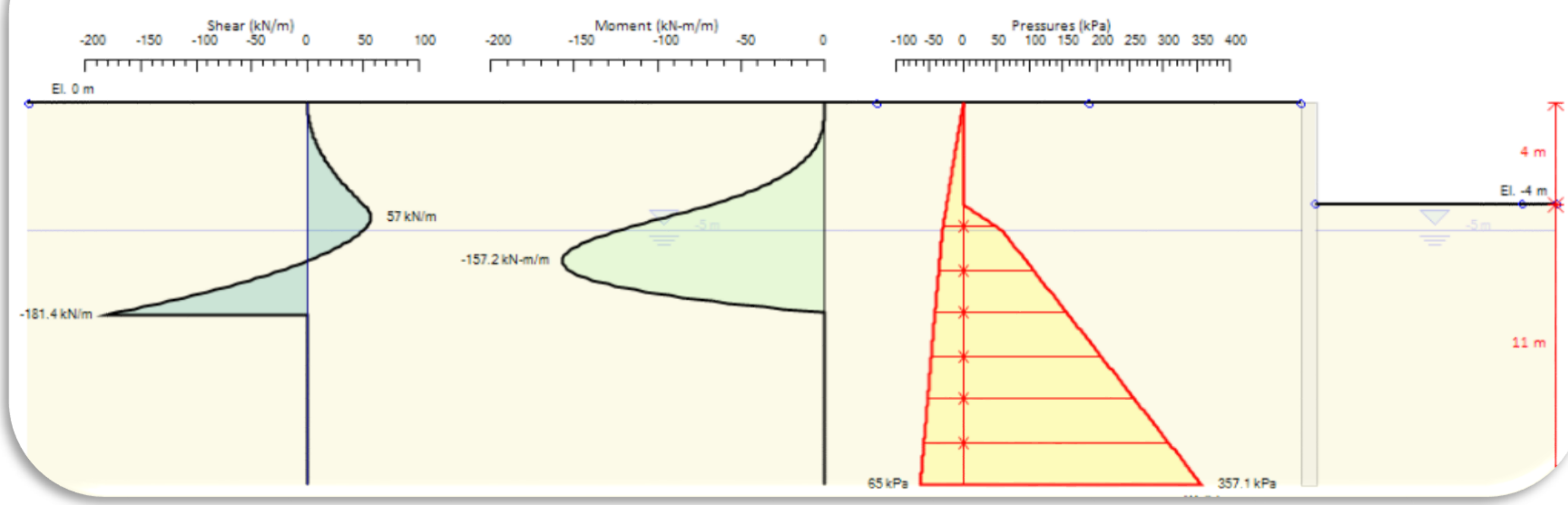
- Free earth method (balance Moment)
- Fixed earth method (balance moment–shear)
- Driving earth pressures: Active
- Resisting pressures: Passive or /Safety Factor

Fixed earth method



8.2 Free earth method

- Balances out moment
- Shear not balanced
- Increase length by 1.2 to get FS 1.0
- Then apply additional safety factors



Left Side El. = 0 FT   Right Side El.= -10 FT   Gen. Water El= -10 FT

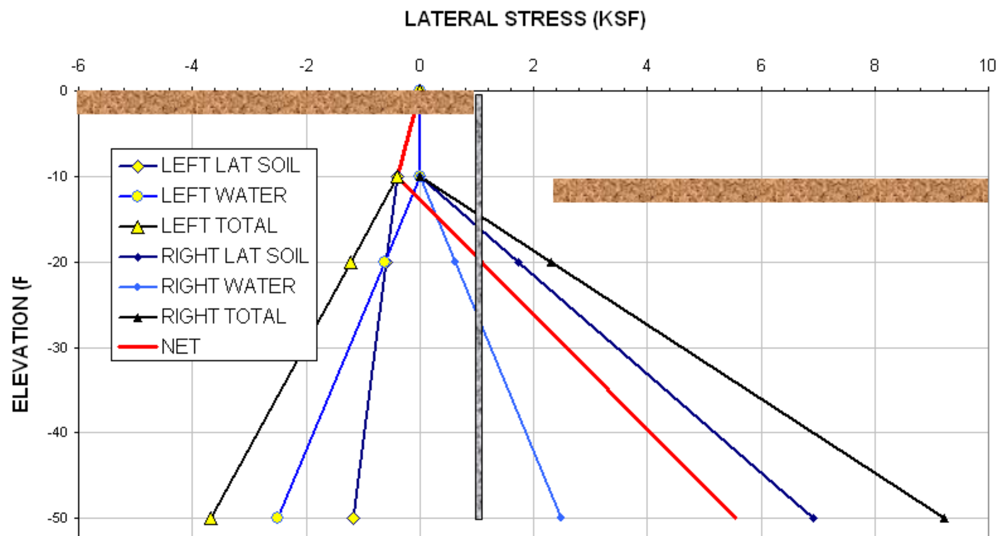
Soil γ= 120 pcf   Friction Angle=30 deg   Water γ= 62.4 pcf

Active on left side ka= 0.333   Passive on right side kp=3

SOIL UNIT WEIGHT (kcf)	WATER UNIT WEIGHT (kcf)	WATER TABLE ELEV. (FT)	Ka	Kp
0.12	0.0624	-10	0.333	3

WATER TABLE ELEV. (FT)
-10

	LEFT EXCAVATION SIDE PRESSURES					RIGHT SIDE PRESSURES					
ELEV. (FT)	TOTAL VERTICAL STRESS (ksf)	WATER PRESSURE (ksf)	EFFECTIVE VERTICAL STRESS (ksf)	LATERAL SOIL STRESS (ksf)	TOTAL LATERAL STRESS	TOTAL VERTICAL STRESS (ksf)	WATER PRESSURE (ksf)	EFFECTIVE VERTICAL STRESS (ksf)	LATERAL SOIL STRESS (ksf)	TOTAL LATERAL STRESS	**NET** (ksf)
0	0	0	0	0	0						0
-10	1.2	0	1.2	-0.4	-0.4	0	0	0	0	0	-0.4
-20	2.4	-0.624	1.776	-0.592	-1.216	1.2	0.624	0.576	1.728	2.304	1.088
-50	6	-2.496	3.504	-1.168	-3.664	4.8	2.496	2.304	6.912	9.216	5.552



LATERAL NET PRESSURES ABOVE SUBGRADE

$kip := 1000lbf$ $ksf := 1\frac{kip}{ft^2}$

$\sigma_{top} := 0ksf$ $\sigma_{bot} := -.4ksf$ $L_c := 10ft$

Lateral force above subgrade $F_1 := (\sigma_{top} + \sigma_{bot}) \cdot L_c \cdot 1\frac{ft}{2}$ $F_1 = -2\,kip$

Centroid to force above subgrade $L_{c1} := \frac{L_c}{3}$ $L_{c1} = 3.333\,ft$

LATERAL NET PRESSURES BELOW SUBGRADE

$\sigma_{sub} := \sigma_{bot}$ $\sigma_{sub} = -0.4\,ksf$

At bottom of wall (El-50) $\sigma_{bw} := 5.552ksf$

Wall length below subgrade $L_{wb} := 40ft$

Passive pressure slope $m_p := \frac{(\sigma_{bw} - \sigma_{sub})}{L_{wb}}$ $m_p = 0.149\frac{ksf}{ft}$

Depth to zero passive pressure from subgrade $EL_o := \left(\frac{-\sigma_{sub}}{m_p}\right)$ $EL_o = 2.688\,ft$

Lateral force from subgrade to Elo $F_2 := \frac{(\sigma_{sub} \cdot EL_o) \cdot 1ft}{2}$ $F_2 = -0.538\,kip$

Now in order to find toe embedment depth for a safety factor of 1, the total net moment must be zero

Assumed depth from Elo to TOE FS 1 Elevation $d_1 := 12.075ft$ ← x 1.2 for FS= 1.0

Sum moments above ELo

$$M_{top} := [F_1 \cdot (L_{c1} + EL_o + d_1)] + \left[F_2 \cdot \left(d_1 + EL_o \cdot \frac{2}{3}\right)\right]$$ $M_{top} = -43.648\,kip \cdot ft$

Lateral net pressure at TOE FS1 Elevation $\sigma_{FS1} := d_1 \cdot m_p$ $\sigma_{FS1} = 1.797\,ksf$

NET Resisting lateral force below Elo $F_3 := \sigma_{FS1} \cdot \frac{d_1 \cdot 1ft}{2}$ $F_3 = 10.848\,kip$

NET Resisting Moment

$M_{BOT} := F_3 \cdot \frac{d_1}{3}$ $M_{BOT} = 43.663\,kip \cdot ft$

TOTAL NET MOMENT $M_{NET} := M_{BOT} + M_{top}$ Which is equal to zero

Elevation at safety factor of 1 $EL_{FS1} := -10ft - EL_o - d_1$ $EL_{FS1} = -24.763\,ft$

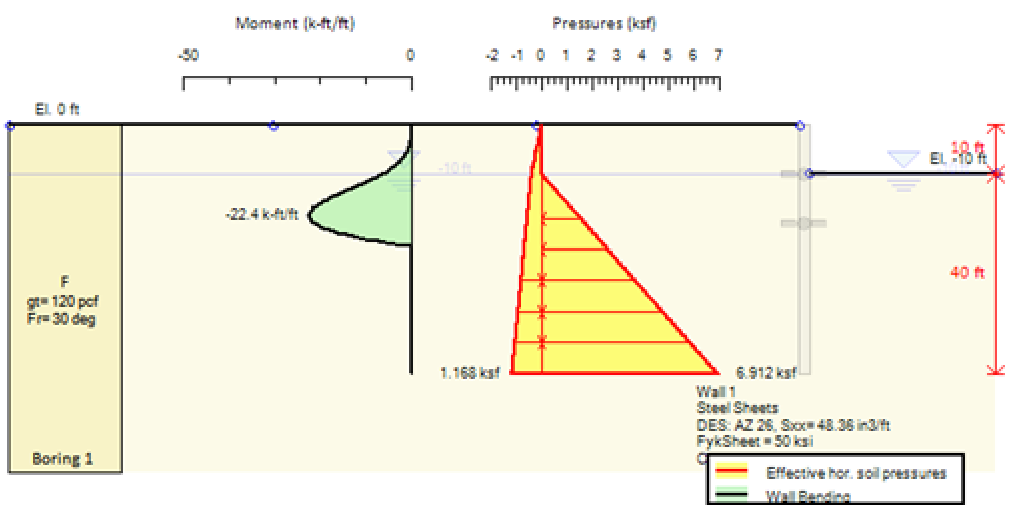
Now find the maximum bending moment. In order to achieve this, the point of zero wall shear must be found first.

Shear to Elo $\quad V_{top} := F_1 + F_2 \quad V_{top} = -2.538\,kip$

Depth to zero shear $\quad d_o := \sqrt{\left(\frac{-2\cdot V_{top}}{m_p\cdot 1ft}\right)} \quad d_o = 5.84\,ft$

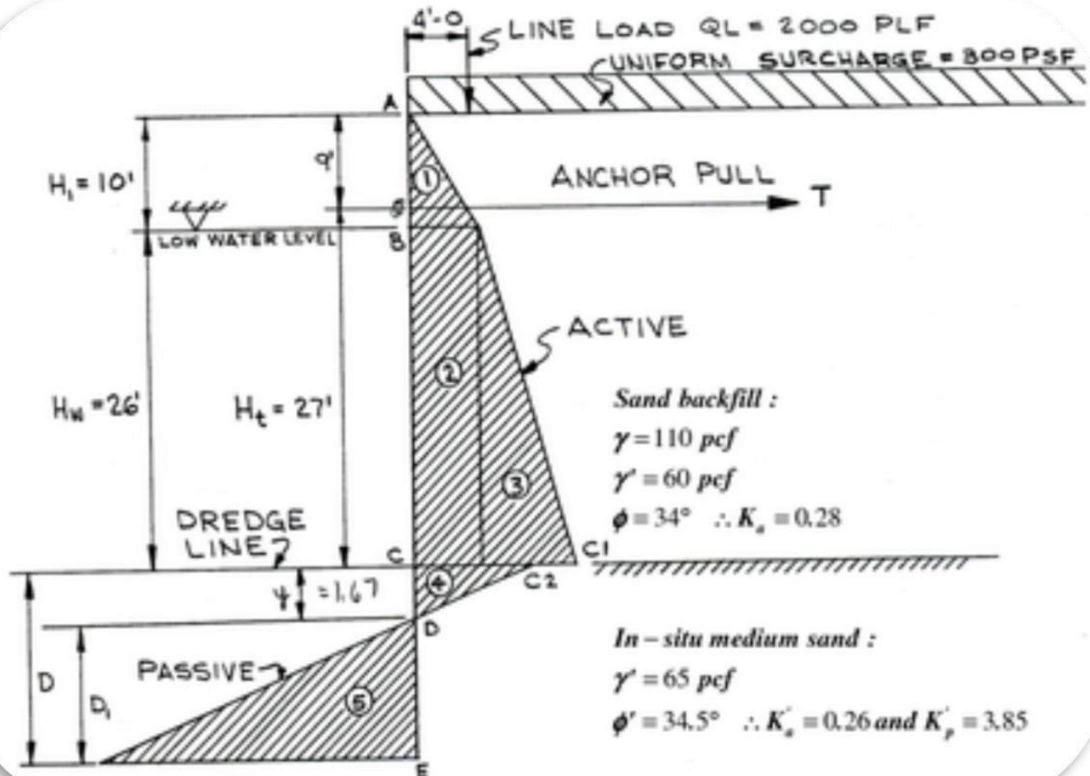
Maximum moment at zero shear

$$M_{max} := \left[F_1\cdot\left(L_{c1} + EL_o + d_o\right)\right] + \left[F_2\cdot\left(d_o + EL_o\cdot\frac{2}{3}\right)\right] - V_{top}\cdot\frac{d_o}{3} \qquad M_{max} = -22.887\,kip\cdot ft$$



8.3 Single support free earth

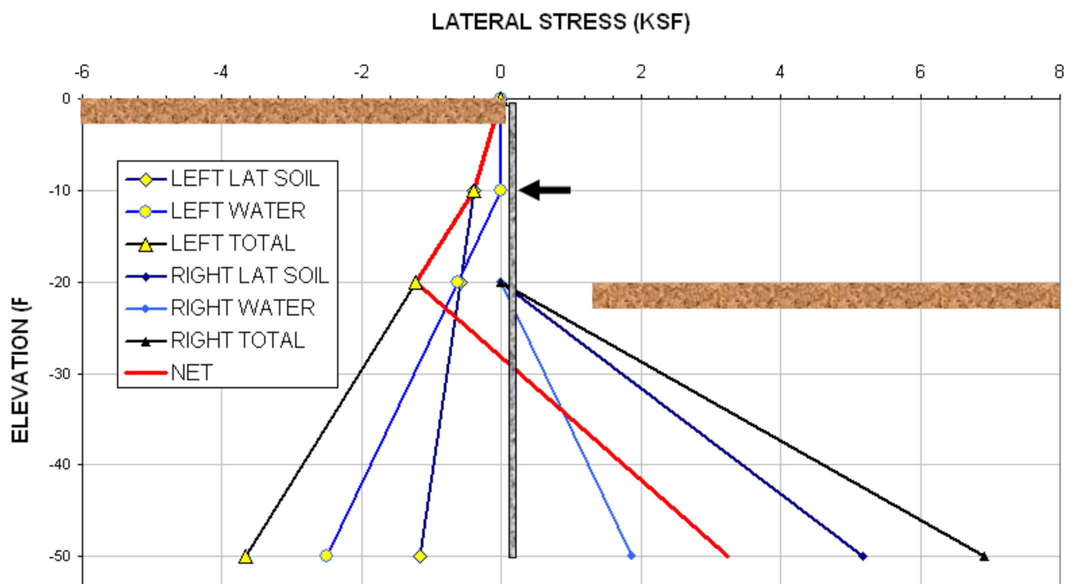
- Sum moments about support level
- All text books show active earth pressures
- Ground anchor prestress?



SOIL UNIT WEIGHT (kcf)	WATER UNIT WEIGHT (kcf)	WATER TABLE ELEV. (FT)	Ka	Kp
0.12	0.0624	-10	0.333	3

WATER TABLE ELEV. (FT)
-20

	LEFT EXCAVATION SIDE PRESSURES					RIGHT SIDE PRESSURES					
ELEV. (FT)	TOTAL VERTICAL STRESS (ksf)	WATER PRESSURE (ksf)	EFFECTIVE VERTICAL STRESS (ksf)	LATERAL SOIL STRESS (ksf)	TOTAL LATERAL STRESS	TOTAL VERTICAL STRESS (ksf)	WATER PRESSURE (ksf)	EFFECTIVE VERTICAL STRESS (ksf)	LATERAL SOIL STRESS (ksf)	TOTAL LATERAL STRESS	**NET** (ksf)
0	0	0	0	0	0						0
-10	1.2	0	1.2	-0.4	-0.4						-0.4
-20	2.4	-0.624	1.776	-0.592	-1.216	0	0	0	0	0	-1.216
-50	6	-2.496	3.504	-1.168	-3.664	3.6	1.872	1.728	5.184	6.912	3.248



LATERAL NET PRESSURES ABOVE SUPPORT

$kip := 1000lbf$ $ksf := 1\frac{kip}{ft^2}$ $\sigma_{top} := 0ksf$ $\sigma_{bot} := -.4ksf$ $L_c := 10ft$

Lateral force above support $F_1 := (\sigma_{top} + \sigma_{bot}) \cdot L_c \cdot 1\frac{ft}{2}$ $F_1 = -2\,kip$

Centroid to force above support $L_{c1} := \frac{L_c}{3}$ $L_{c1} = 3.333\,ft$

Moments above support $M_1 := L_{c1} \cdot F_1$ $M_1 = -6.667\,kip \cdot ft$

LATERAL NET PRESSURES BELOW SUPPORT AND ABOVE SUBGRADE EL-20ft

$\sigma_{sub} := -1.216ksf$ $L_2 := 10ft$ DEPTH FROM SUPPORT TO SUBGRADE

Rectangular portion force $F_{rect} := \sigma_{bot} \cdot L_2 \cdot 1ft$ $F_{rect} = -4\,kip$

Moment about support $M_{2RECT} := -F_{rect} \cdot \frac{L_2}{2}$ $M_{2RECT} = 20\,kip \cdot ft$

Triancular portion of force $F_{tri} := (\sigma_{sub} - \sigma_{bot}) \cdot \frac{L_2 \cdot 1ft}{2}$ $F_{tri} = -4.08\,kip$

Moment about support $M_{2TRI} := -F_{tri} \cdot L_2 \cdot \frac{2}{3}$ $M_{2TRI} = 27.2\,kip \cdot ft$

$M_2 := M_{2TRI} + M_{2RECT}$ $M_2 = 47.2\,kip \cdot ft$

LATERAL NET PRESSURES BELOW SUBGRADE EL-20ft

At bottom of wall (El-50) $\sigma_{bw} := 3.248ksf$ Wall length below subgrade $L_{wb} := 30ft$

Passive pressure slope $m_p := \frac{(\sigma_{bw} - \sigma_{sub})}{L_{wb}}$ $m_p = 0.149\frac{ksf}{ft}$

Depth to zero passive pressure from subgrade $EL_o := \left(\frac{-\sigma_{sub}}{m_p}\right)$ $EL_o = 8.172\,ft$

Lateral force from subgrade to Elo $F_3 := \frac{(\sigma_{sub} \cdot EL_o) \cdot 1ft}{2}$ $F_3 = -4.969\,kip$

Moment about support $M_3 := -\left(L_2 + \frac{EL_o}{3}\right) \cdot F_3$ $M_3 = 63.221\,kip \cdot ft$

Net moment above Elo $M_{NET1} := M_1 + M_2 + M_3$ $M_{NET1} = 103.754\,kip \cdot ft$

To find toe embedment depth for a safety factor of 1, the total net moment must be zero

Assume depth to FS1 below ELo $d_1 := 7.7323ft$

Pressure at d1 $\sigma_{FS1} := m_p \cdot d_1$ $\sigma_{FS1} = 1.151\,ksf$ $F_4 := \sigma_{FS1} \cdot d_1 \cdot 1\frac{ft}{2}$ $F_4 = 4.448\,kip$

Moment about support $M_4 := -\left(L_2 + EL_o + d_1 \cdot \frac{2}{3}\right) \cdot F_4$ $M_4 = -103.764\,kip \cdot ft$

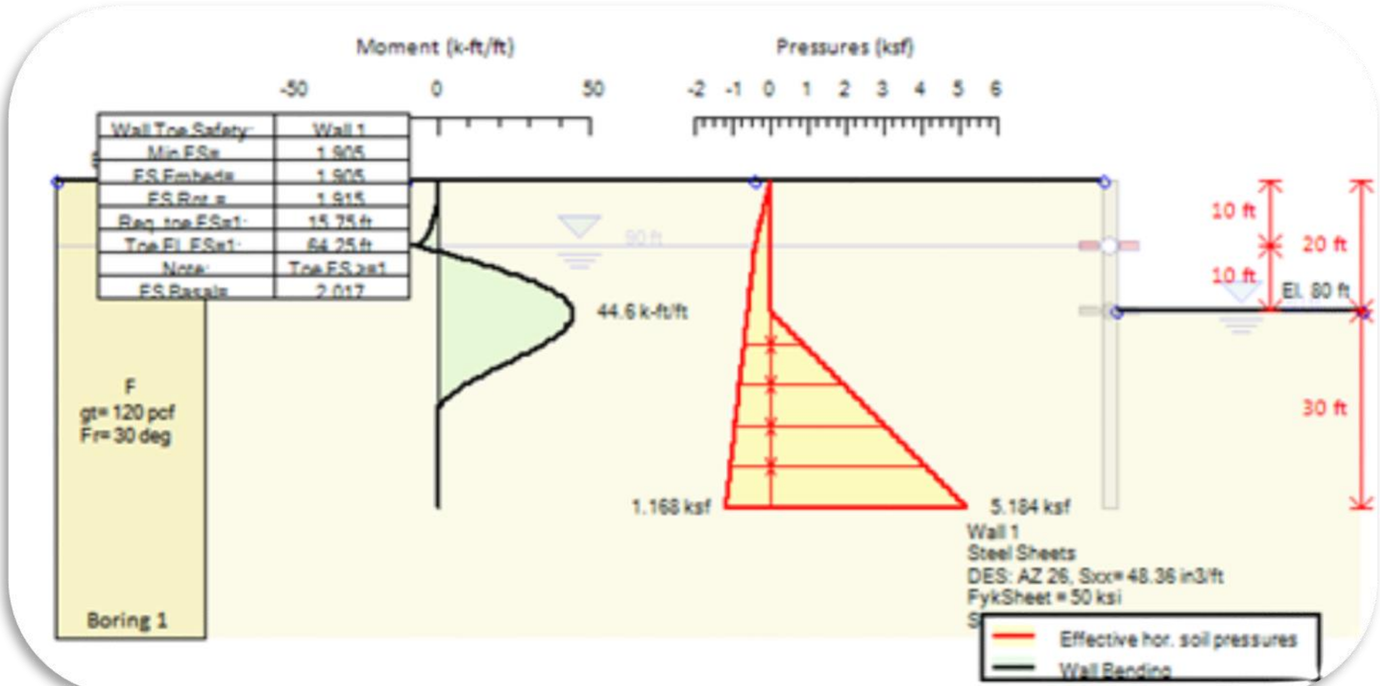
NET MOMENT $M_{NET} := M_{NET1} + M_4$ $M_{NET} = -0.01\,kip \cdot ft$

Elevation at FS 1 $EL_{FS1} := -20ft - EL_o - d_1$ $EL_{FS1} = -35.904\,ft$

In free earth method for walls with one support levels, both shear and wall moment balance out at base of wall.

Length does not need to be Increased for FS=1.0 to be achieved

Sample output from software

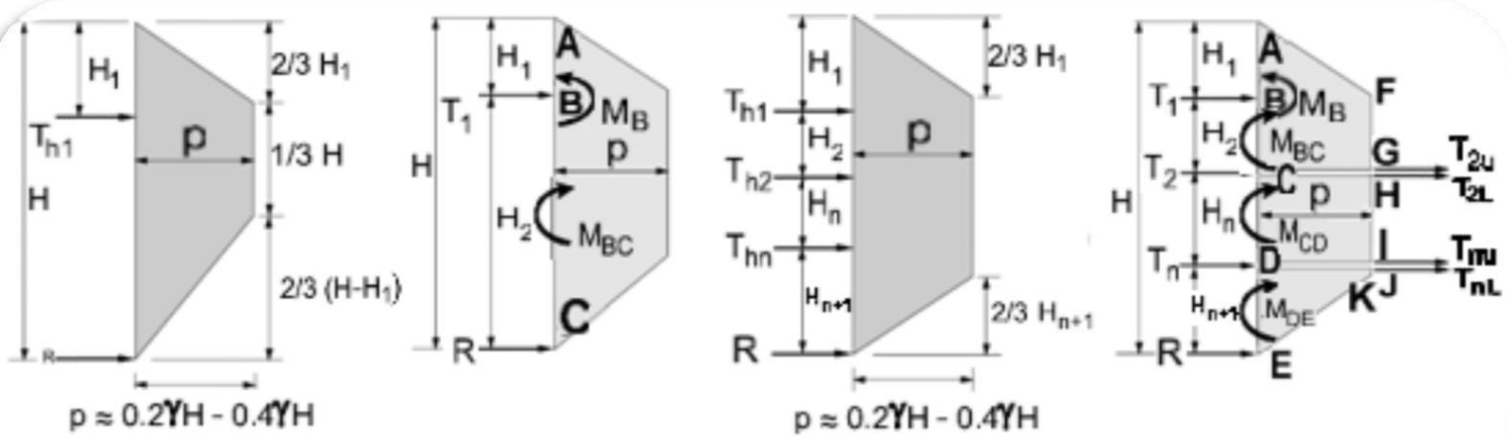


8.4.1 Apparent earth pressures

- Earth pressures back calculated from Strut loads.
- Peck 1969, early excavations in Chicago.
- Private discussion with Dr. Peck, gamma is effective, water to be added separately.
- Reaction at subgrade?

8.4.2 Apparent earth pressures

- Envelopes captured maximum force from all stages
- Wall moments were almost never measured!
- Wall moment recommendations may not be reasonable!

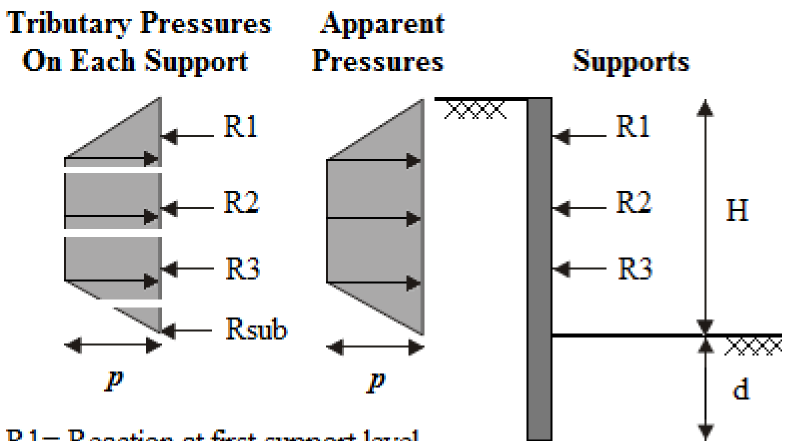


a. Walls with one support level

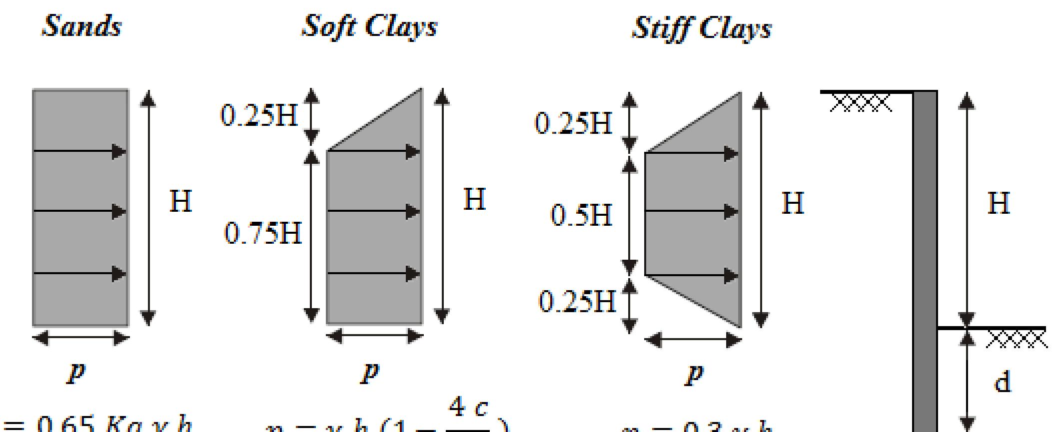
b. Walls with multiple supports

8.4.3 Apparent earth pressures

Peck, 1969



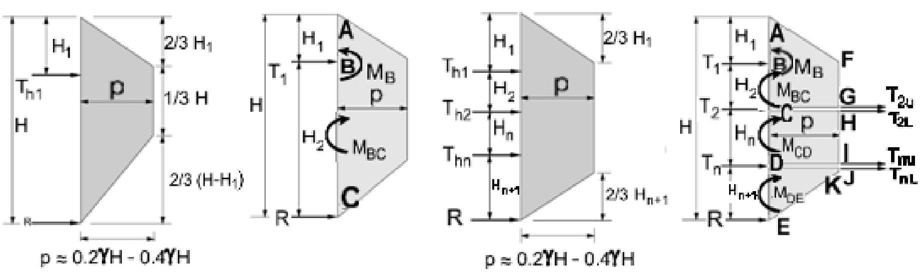
R1= Reaction at first support level
Rsub = Apparent reaction at subgrade level



$p = 0.65\,Ka\,\gamma\,h$

$p = \gamma\,h\,(1 - \frac{4\,c}{\gamma\,h})$ $for\ \frac{4\,c}{\gamma\,h} > 1$

$p = 0.3\,\gamma\,h$ (Range 0.2 to 0.4)



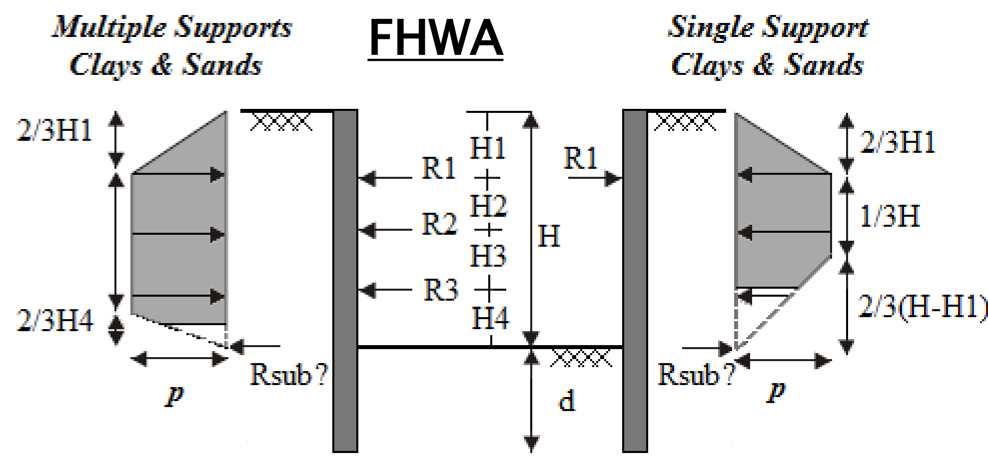
$p \approx 0.2\gamma H - 0.4\gamma H$

a. Walls with one support level

$p \approx 0.2\gamma H - 0.4\gamma H$

b. Walls with multiple supports

FHWA



Sands: Total Load = $0.65\,Ka\,\gamma\,h^2$

Clays: $p = 0.2\,\gamma\,h$ to $0.4\,\gamma\,h$

Rsub = Apparent virtual reaction at subgrade

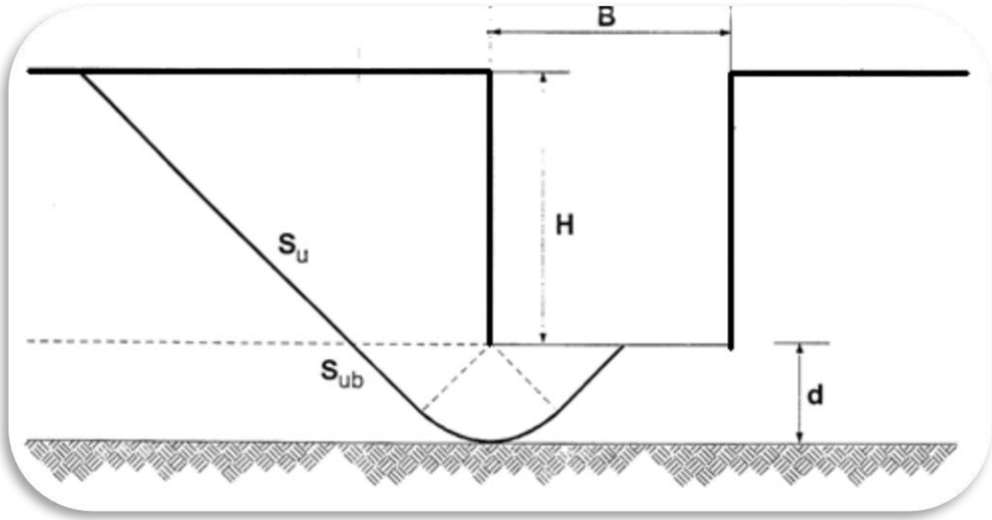
8.4.4 FHWA – Basal failure (Ns>6)

$$Ns = \frac{\gamma_{total}\, H}{S_u}$$

Where m=1 according to Henkel (1971). The total load is then taken as:

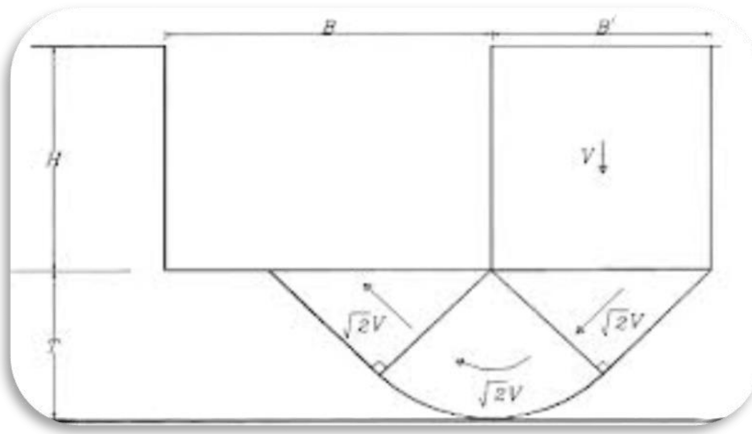
$$K_A = 1 - m \cdot \frac{4\, S_u}{\gamma\, H} + 2\sqrt{2}\frac{d}{H}\left(1 - \frac{5.14\, S_{ub}}{\gamma\, H}\right)$$

$$P = 0.5\, K_A\, \gamma\, H^2$$



Henkel's mechanism of base failure

8.4.5 Basal stability



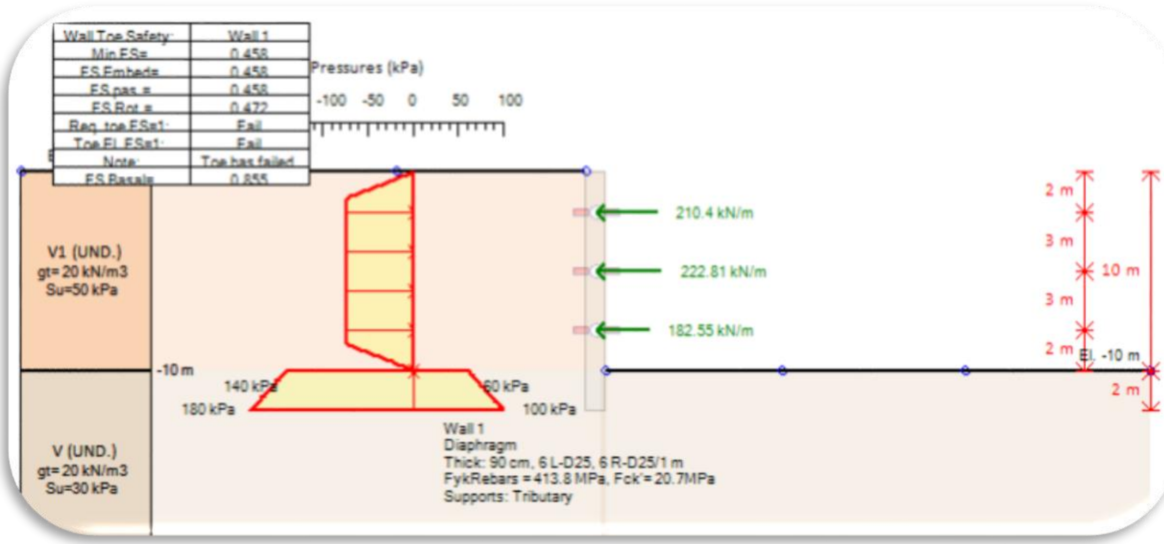
$FS = \dfrac{N_c s_{ub}}{\gamma - s_{ub}/D}$	For $D<(\sqrt{2}/2)B$
$FS = \dfrac{N_c s_{ub}}{\gamma - \dfrac{2 s_{uu}}{\sqrt{2}B}}$	For $D>(\sqrt{2}/2)B$
s_{ub} = undrained strength of basal clay s_{uu} = undrained strength of clay above the excavation grade B = Breadth of excavation D = Depth from excavation grade to firm stratum $N_c = 5.4$	

8.4.5 Design example soft clays

- 10m excavation in clay
- Analyze with FHWA

Clay 1: From 0 to 10m depth,
Su = 50 kPa γ= 20 kN/m^3

Clay 2: From 10m depth and below
Su = 30 kPa γ = 20 kN/m^3



8.4.5 Design example soft clays

The total vertical stress at the excavation subgrade is:

$$\sigma_v' = 20\ kN/m^3 \times 10m = 200\ kPa$$

The basal stability safety factor is then:

FS= 5.7 x 30 kPa/ 200 kPa = 0.855 (verified from Fig. 2.10)

$$Ns = \frac{\gamma_{total} H}{S_u} = \frac{20 \, x \, 10}{30} = 6.67$$

Then according to Henkel Ka is calculated as (m=1):

$$K_A = 1 - \frac{4\,S_u}{\gamma\,H} + 2\sqrt{2}\frac{d}{H}\left(1 - \frac{5.14\,S_{ub}}{\gamma\,H}\right)$$

$$K_A = 1 - \frac{4\,x\,50kPa}{200\,kPa} + 2\sqrt{2}\frac{10m}{10m}\left(1 - \frac{5.14\,x\,30\,kPa}{200\,kPa}\right) = 0.647$$

The total thrust above the excavation is then: $P_{total} = 0.5\ K_A\ \sigma_v' \times H$ = 647 kN/m

The maximum earth pressure ordinate is then:

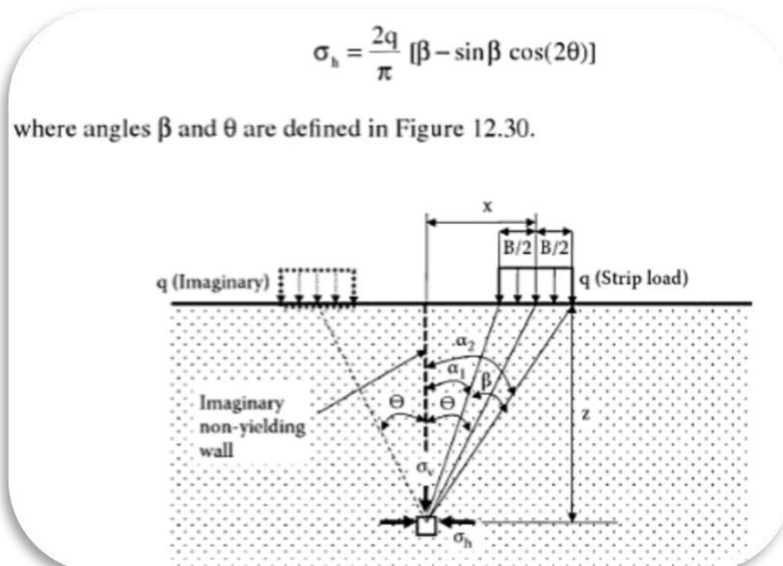
p= 2 x Load /{2 H – 2(H_1 + H_{n+1})/3)} = 2 x 647 kN/m /{2 x 10m -2 x (2m +2m)/3}= 74.65 kPa

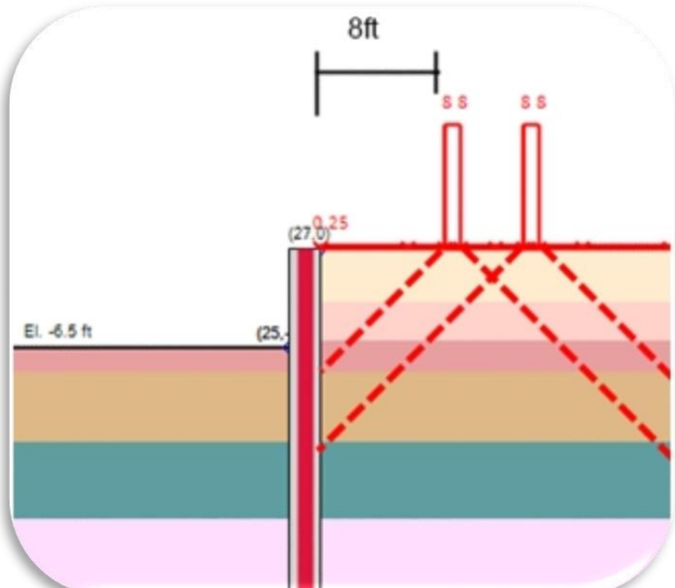
8.4.6 Support reactions

- Middle support most critical.
  Tributary area method
  3m x 74.65 kN/m2 = 223.95 kN/m
- Wall bending simple moment?
  $M = wL^2/8$ = 83.98 kN-m/m

8.5.1 Surcharges

- Theory of elasticity
- Rigid walls with Boussinesq, x 2
- Distribution angle on vertical stress

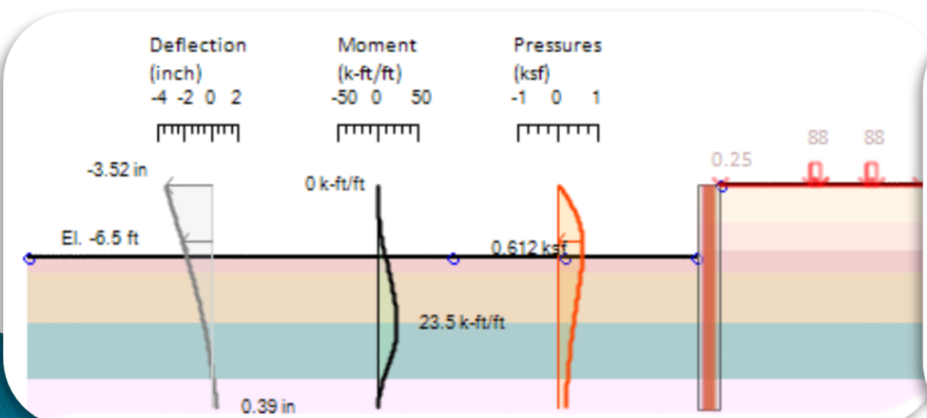


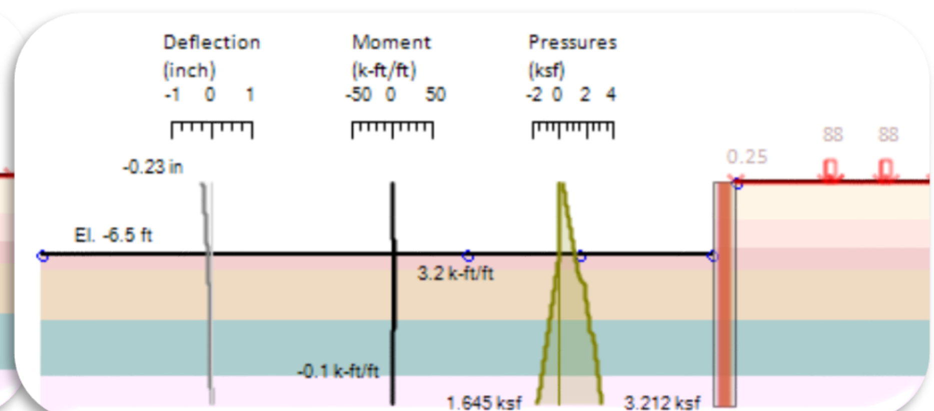


8.5.2 Surcharge example

- 6.5ft excavation (2.0m)
- Train loads 11ft back
- Compare results

Case	Wall Displacement (in)	Wall Moment (k-ft/ft)
Rigid conditions m=2	3.52	23.5
Flexible conditions m=1	1.95	15.95
Distribution angle	0.23	3.29



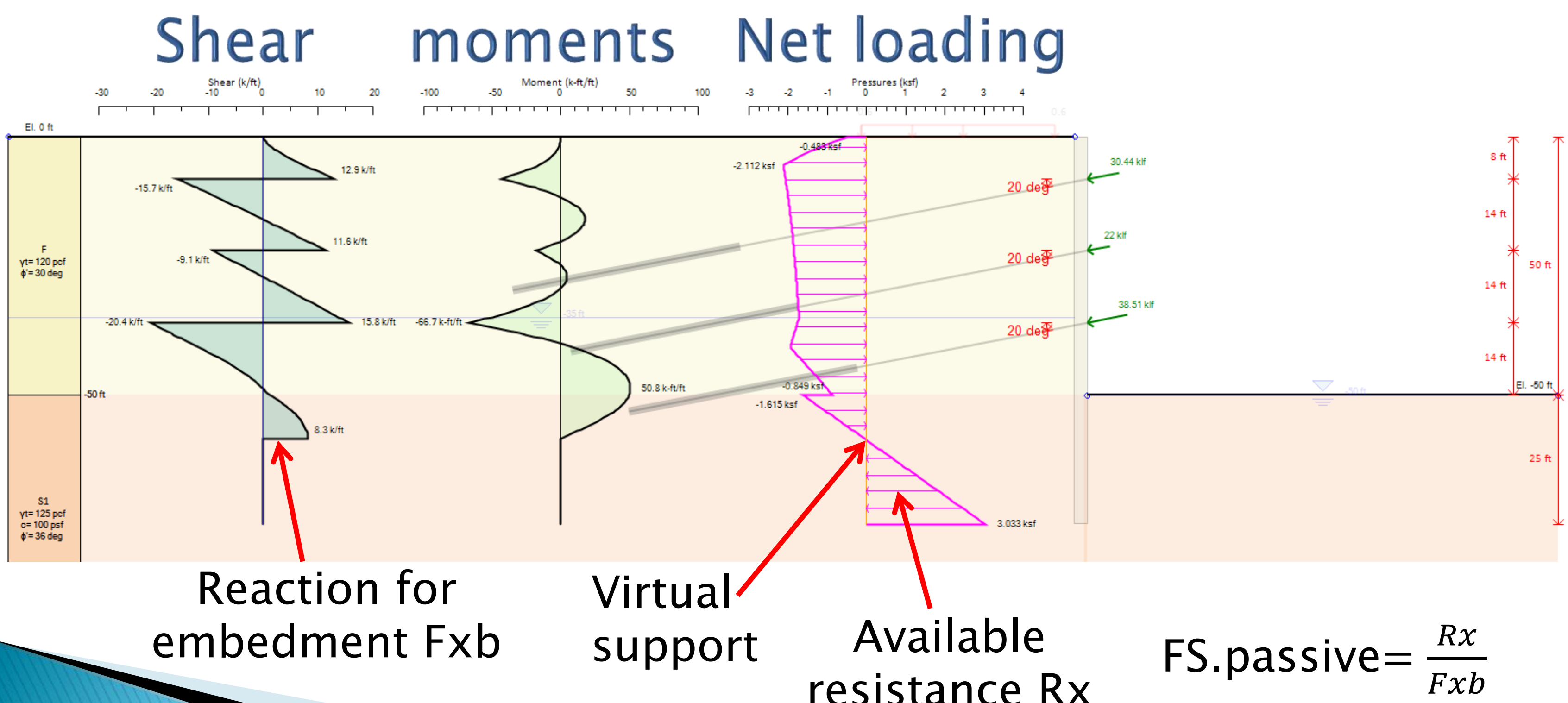


9. Beam analysis – multiple supports

- Blum's method
- FHWA method with simple spans (GEC-4)
- Mix between FHWA and Blum's
- CALTRANS Trenching and Shoring Manual

9.1 Blum's method

- Pinned supports – continuous beam
- Point of zero net soil shear below subgrade.
- Use point of zero shear as a virtual support.

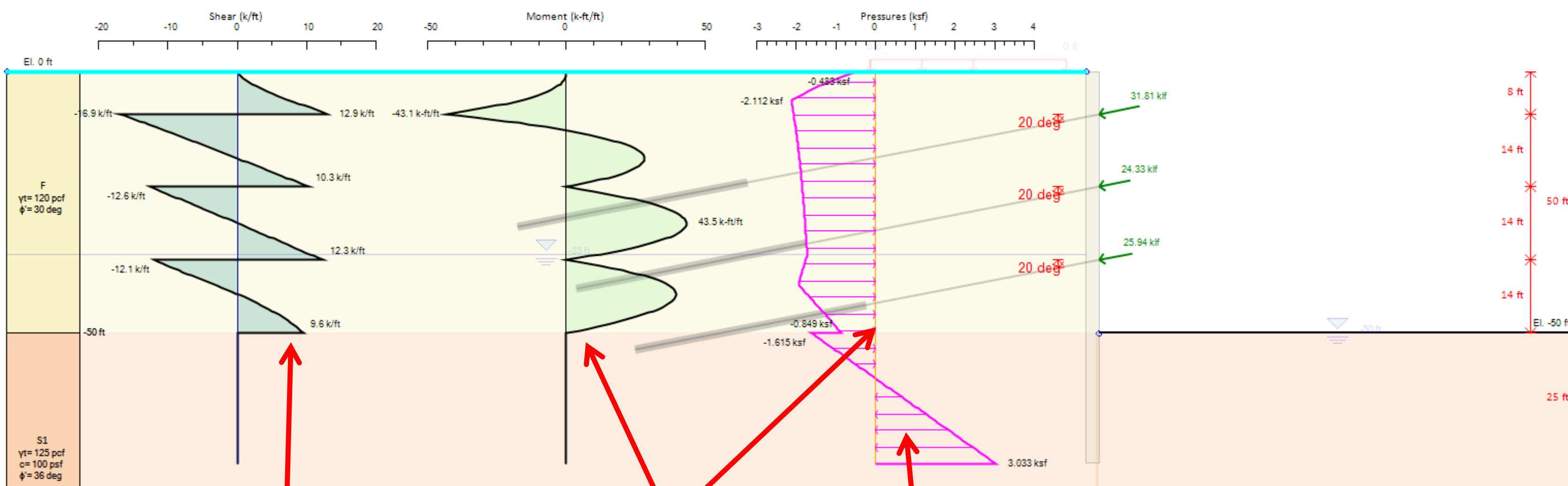


$$\text{FS.passive} = \frac{Rx}{Fxb}$$

9.2 FHWA Simple Span Approach

- Pin support at excavation base, simple spans

Shear moments Net loading



Reaction for embedment Fxb

Virtual support

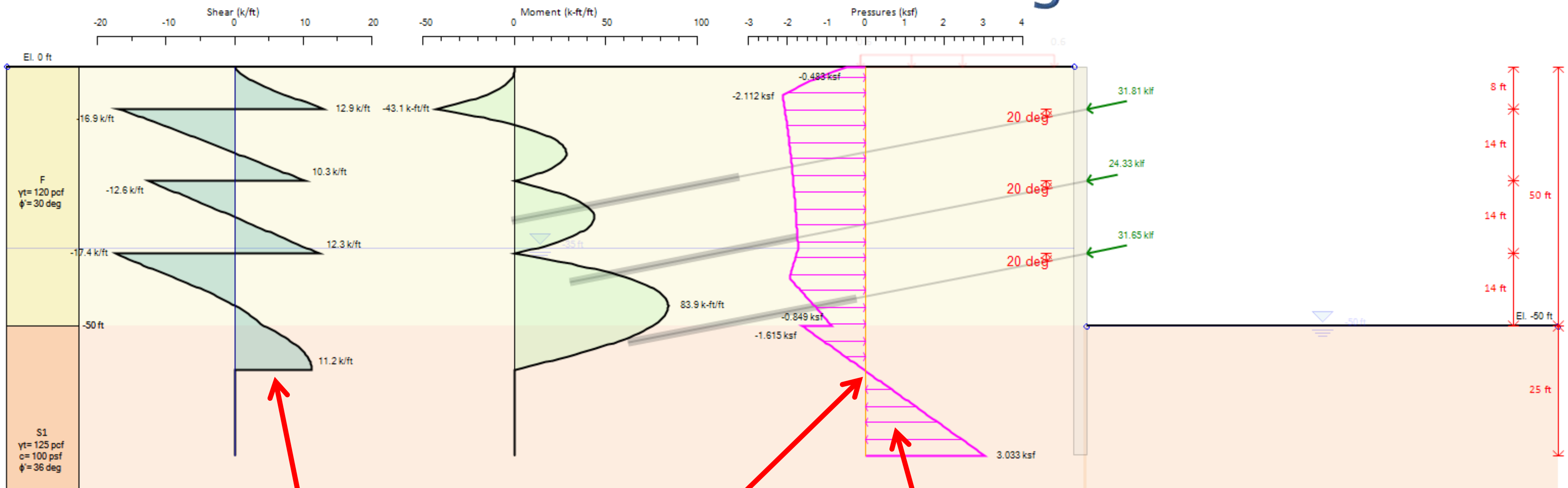
Available resistance Rx

$$FS.passive = \frac{Rx}{Fxb}$$

9.3 Modified FHWA-Blum

- Pinned supports – simple span
- Point of zero net soil shear below subgrade

Shear moments Net loading



Reaction for embedment Fxb

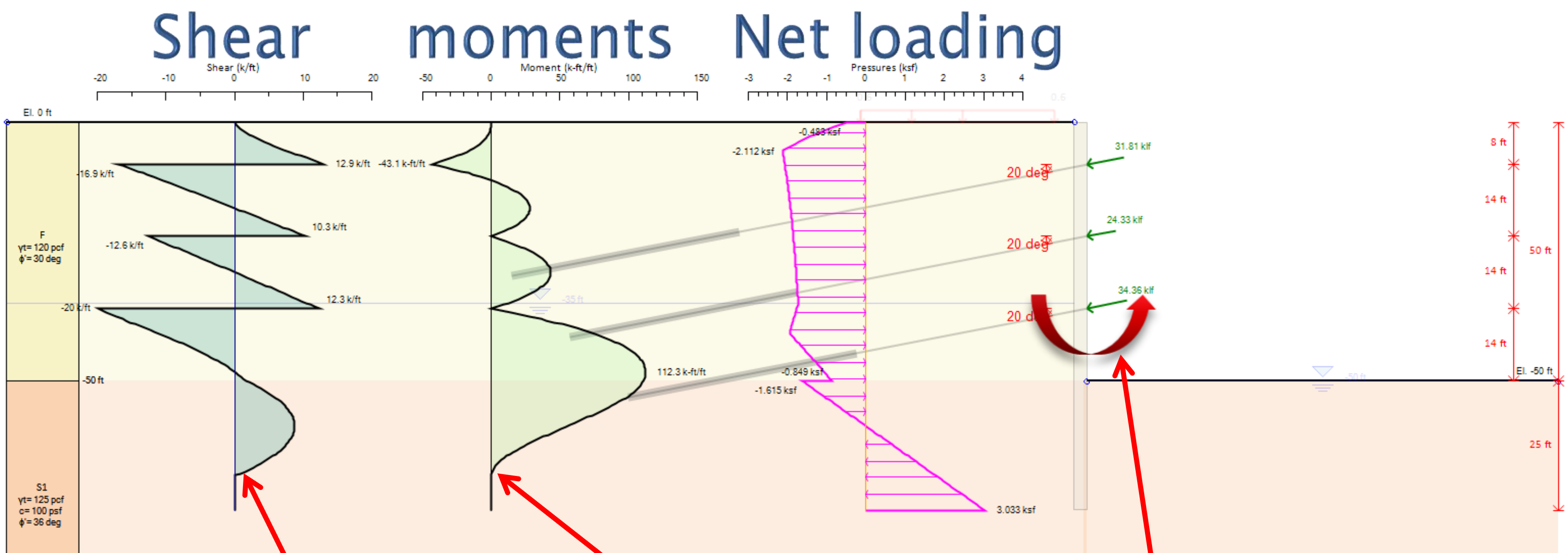
Virtual support

Available resistance Rx

$$FS.passive = \frac{Rx}{Fxb}$$

9.4 Caltrans method

- Pinned supports – simple span
- Base at point of zero moment below bottom support
- Shears and moments balance out



No Reaction embedment

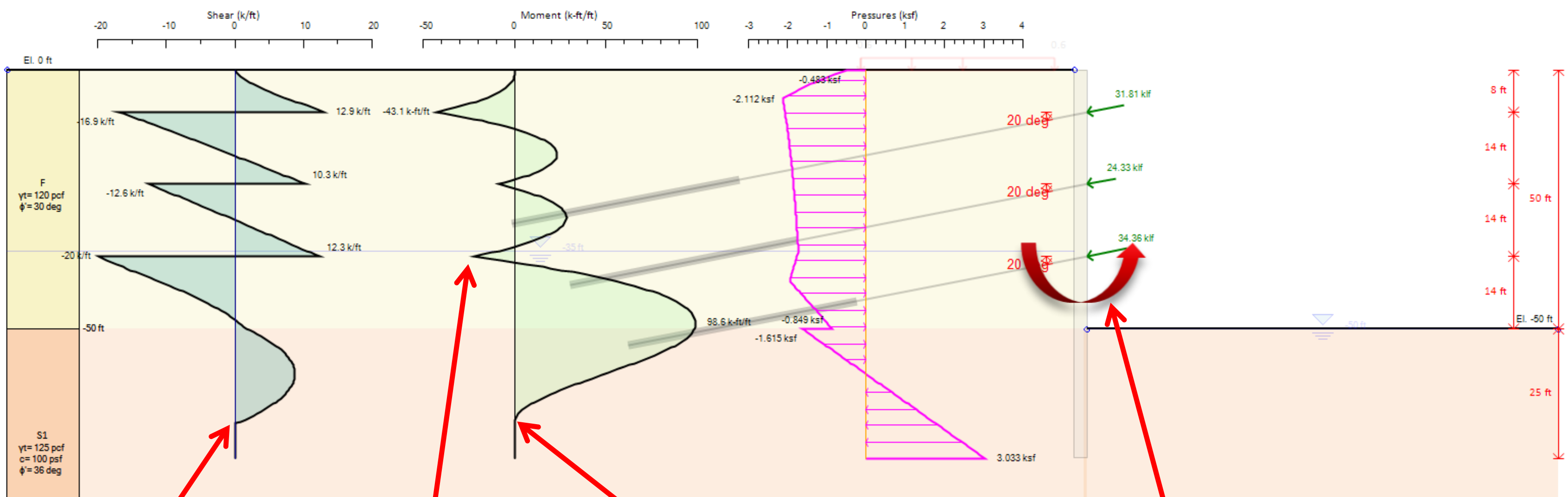
Virtual support

$$\text{FS.rotation} = \frac{Mresist}{Mdrive}$$

9.5 Caltrans & negative moments

- Simple span may be very conservative
- Assume negative moments (20% of simple span)

Shear moments Net loading



No Reaction embedment

Negative Moment

Virtual support

$$\text{FS.rotation} = \frac{Mresist}{Mdrive}$$

9.6 How methods compare

Static loading final excavation

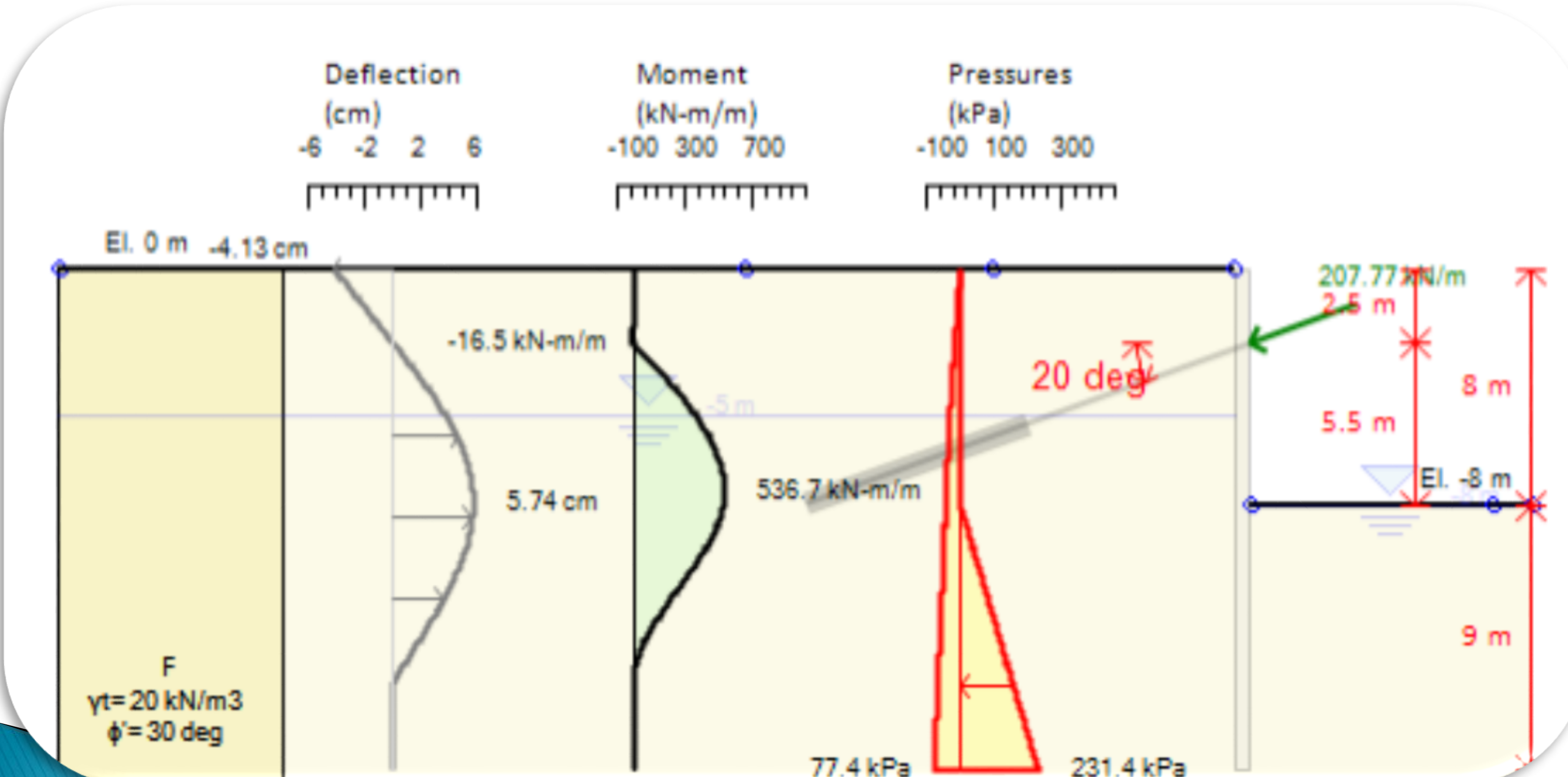
	Blums's method	FHWA Simple span	FHWA Mixed Blum	CALTRANS Method	CALTRANS – negative	Nonlinear analysis*
Maximum support reaction (kips/ft)	33.68	23.91	27.66	30.08	30.08	30 – 31.8
Maximum Moment (kips/ft)	58.25	36.78	74.29	99.41	87.45	65 – 86
Maximum Shear (kips/ft)	18.13	13.14	15.49	17.77	17.77	17.4 – 20

Seismic 0.1g at last stage

	Blums's method	FHWA Simple span	FHWA Mixed Blum	CALTRANS method	CALTRANS – negative	Nonlinear analysis*
Maximum support reaction (kips/ft)	38.51	31.81	31.81	34.36	34.36	31.6 – 34.6
Maximum Moment (kips/ft)	66.74	43.46	83.95	112.33	98.63	76.9–101.5
Maximum Shear (kips/ft)	20.41	16.94	17.45	20	20	19.5 – 22.2

9.7.1 Anchor Prestress effect

- Compared LEM with B.E.F. (NL)
- LEM: Active, FHWA, Peck
- Examine 100%, 110%, 120% Ka prestress



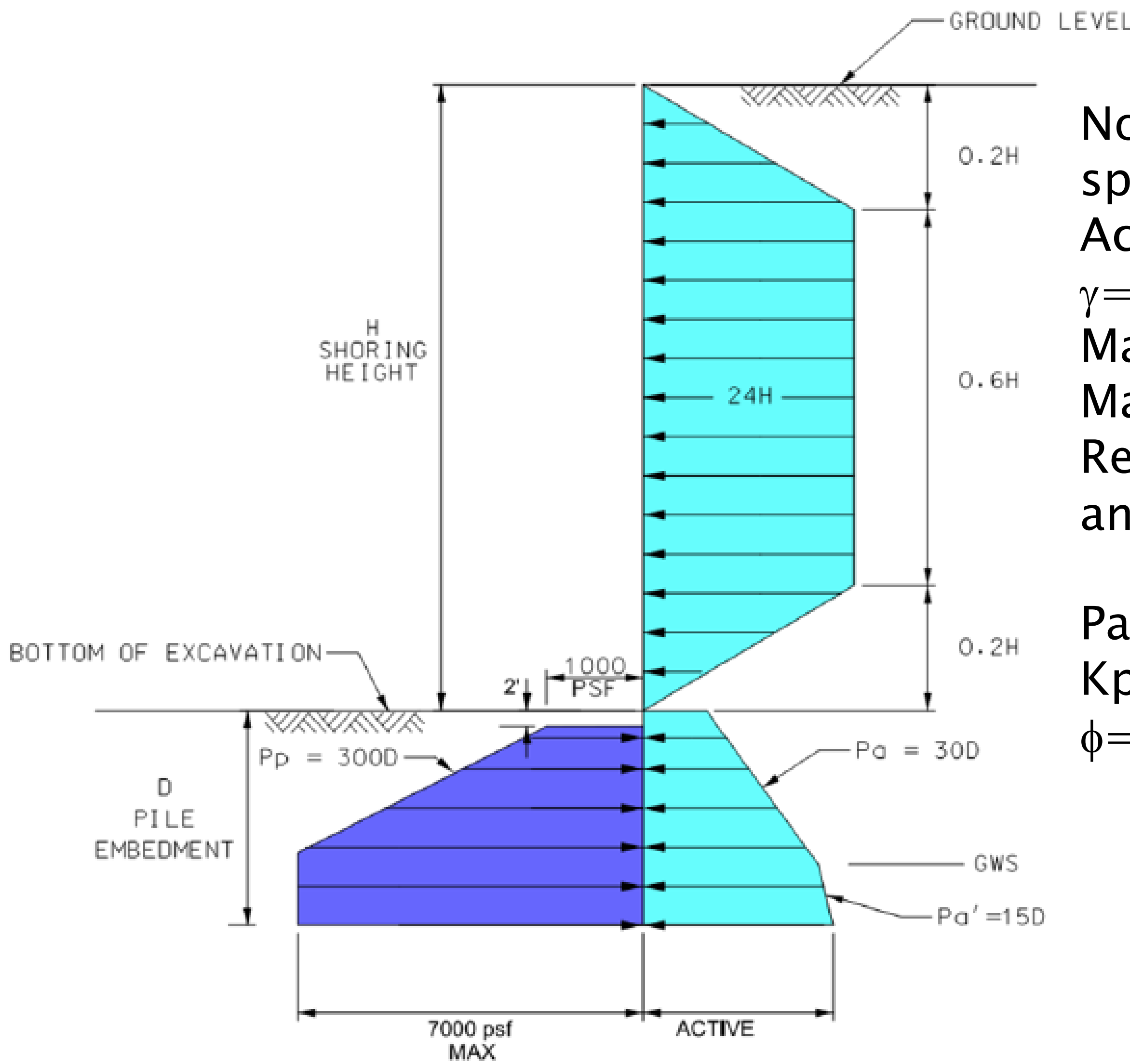
9.7.2 Results

Examined case	Wall Dx (cm)	Wall Moment (kN-m/m)	Max Support Reaction (kN/m)	Toe FS Rotation (LEM)	Toe FS Length (LEM)	FS Mobilized Passive (NL)
LEM-Active	5.74	536.7	207.9	1.633	1.494	N/A
LEM-FHWA	4.02	386.4	270.2	1.698	1.56	N/A
LEM-Peck	4.43	433.7	263.1	1.676	1.537	N/A
NL - 100% Active	6.68	467.2	269.6	N/A	N/A	1.462
NL - 110% Active	6.47	463.3	282.2	N/A	N/A	1.465
NL -120% Active	6.28	460.02	294.9	N/A	N/A	1.468

10.1 No earth pressures

- Pressure are not a property
- Construction staging
- Wall–to–soil friction
- Support prestress
- Wall deflections
- Surface profile
- You think you are safe!

10.2 Example of confusing specs



Not physically possible specifications
Active pressures
γ=125 pcf (19kN/m3)
Ma.dry = 30 D
Ma.wet = 15 D
Results in Ka= 0.24
and ϕ= 37.8 degrees

Passive slope produces
Kp= 2.4, thus
ϕ= 24.32 degrees

11. Conclusions

- Use at least two different analysis methods.
- Understand soil and project needs.
- Soil and structure interact –

  Lateral earth pressures are not a property

12. What is next

- **Next week: March 3, 4, 5, 6**
  Second series of webinars:
  Design codes: ASD, LRFD, Eurocode 7
  Worked out examples.
- Third week: March 10, 11, 12
  Optimization of excavations

Thank you!

For attending this webinar.

dimitrios@deepexcavation.com

Design example available at:

http://www.deepexcavation.com/en/50ft-deep-excavation-example

Connect on LinkedIn

DEEP EXCAVATION

RELIABLE GEOEXPERTISE